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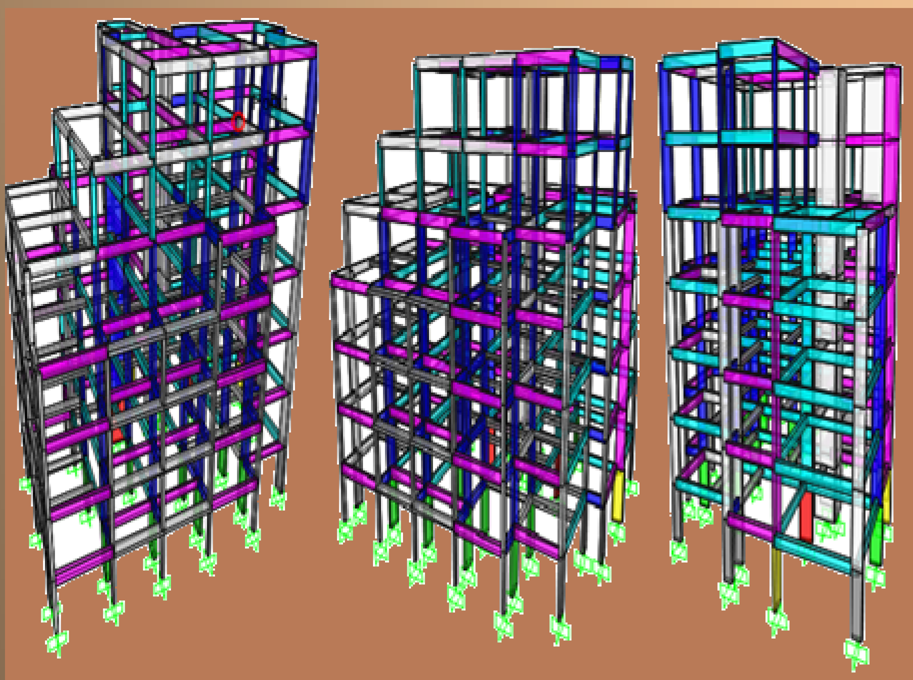
"Analysis and Design of Earthquake Resistant Structures" (ADERS)

Seismic Capacity Assessment and Retrofitting of Reinforced Concrete Building



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CERTIFICATE

This is to certify that the work presented in thesis entitled “Seismic Evaluation of Reinforced Concrete Buildings” submitted by Filippou Christiana in partial fulfillment of requirements for the award of degree of Masters Analysis and design at National Technical University of Athens (NTUA).

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ABSTRACT

The aim of this master thesis is the safety assessment of an existing multistory building and the examination of different reinforcing scenarios of its concrete frame. The building studied is a six storey residential building in Amathounta area in the city of Limassol, Cyprus. The structure was built in 1980. It is constructed from reinforced concrete according to the early Cyprus National Codes as has already been pointed out.

Linear and nonlinear analyses were used for the capacity assessment of the construction. These two different techniques were compared giving an insight to the pros and cons of each method. In this thesis linear methods that were applied to the structure are analyzed. Modal analysis results, modal response spectrum and linear time history analysis methods are presented. The results from each technique are compared in order to acquire the differences among the analysis methods.

Moreover the current study deals with the evaluation of reinforced concrete buildings using inelastic method (Pushover analysis). Capacity curve, which is load-deformation plot, is the output of pushover analysis. As, pushover analysis is non-linear static analysis, so the load-deformation curve can be obtained from Sap2000. It is software which is used to perform the non-linear static pushover analysis. The analysis of the structure showed the need for additional reinforcements to the original concrete frame, in order to improve its seismic behavior.

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1. Introduction

1.1 Seismic activity of Cyprus

Cyprus lies in the second largest earthquake-stricken zone of the earth, but in a relatively less active sector. The level of the seismic activity in the Cyprus region is significantly lower than that in Greece and Turkey. This zone stretches from the Atlantic Ocean across the Mediterranean Basin, through Greece, Turkey, Iran, and India as far as the Pacific Ocean. The energy released by the earthquakes in this zone represents 15% of the universal seismic energy. However, many destructive earthquakes have struck Cyprus over its long history and many of its towns and villages (notably Paphos, Salamina, Kitio, Amathounta, Kourio and Nicosia) have been destroyed by strong earthquakes. In the history of the island there have actually been a few strong earthquakes that have managed to destroy some of the islands cities. Historically, only the most significant earthquakes have been recorded, whilst for recent years a more complete record is available. Between 1500 AD and the present there were 30 destructive earthquakes of intensity 8 or above on the Mercali scale.

During the last century lot of earthquakes hit Cyprus. One of the worse was take place at the 10 of September of 1953 with surface magnitude 6.1.The villages of Stroumpi, Axylou, Kithasi, Lapithiou and Phasoula were totally destroyed. Damage was mainly caused by landslides and ground cracking. Within a few seconds 1600 houses were totally ruined and 10,000 buildings suffered serious damage. Causalities were limited because most people were out in the fields at the time the earthquake occurred. In Limassol the shock caused extensive damage, where it triggered soil liquefaction of beach deposits on the seashore. The earthquake was associated with a small tsunami along the coast of Paphos.

The worst earthquake in the modern history of Cyprus took place at 9 of October at 1996. It had 6.5 surface magnitude and it was located in the southwest of the island. It caused panic in the districts of Pafos, Lemesos, Lefkosia, Larnaca and Ammochostos. Two people lost their lives and 20 were slightly injured. There were damage reports especially in Pafos and Lemesos.

1.2General

Due to all above, the island had economic and social impacts. The human and material losses are huge based on the failures in most buildings that were designed inadequately against the earthquake actions. In nowadays it is crucial to improve the scholars for shielding the constructions against the horizontal actions.

A major problem in our country is the fact that the majority of existing buildings designed and manufactured mainly in the 60s and 70s, when there was intense reconstruction mainly in urban centers due to the creation of the Republic of Cyprus. As a result the buildings are significantly lagging behind in terms of seismic aptitude compared with modern buildings. However, the complete replacement of all these structures with new structures, according to the modern anti-seismic regulations, it is impossible due to economic and social factors. Thus the need for retrofitting in existing constructions, led to the preparation of relevant regulations. These regulations established criteria for assessing the capacity of these existing buildings and implemented rules for the seismic design.

All over the world, the building stock, sometime during it lifetime needs maintenance, repair and upgrading. Moreover, in the light of our current knowledge and of modern codes, the majority of buildings are substandard and deficient. This is happening mainly in earthquake-prone regions, as seismic design of structures is relatively recent even in those regions.

Although today and for the next years most of the seismic threat to human life and property loss comes from the existing buildings, the emphasis of earthquake engineering research and of code-writing efforts has been, and still is, on new construction. This is probably an optimal solution from the socioeconomic point of view, provided that the rate of occurrence of moderate to strong earthquakes is much lower than the attrition rate of old buildings.

1.2.1Eurocodes

Eurocodes are expected to:

1. Improve the functioning of the single market for products and engineering services by removing obstacles arising from different nationality codified practices for the assessment of structural reliability.
2. Improve the competitiveness of European construction industry and the professionals and industries connected to it in countries outside the European Union.

In this case study the Eurocodes (EC) 1, 2 and 8 are mainly used. Eurocode 1 is referred to the actions on constructions. Eurocode 2 is for the Design of concrete structures. Eurocode 1 gives the design guidance and actions for the structural design of buildings and civil engineering works including some geotechnical aspects for the following subjects:

- Densities of construction materials and stored materials
- Self-weight of construction works
- Imposed loads for buildings

Eurocode 2 applies to the design of buildings and civil engineering works in plain, reinforced and pre – stressed concrete. It complies with the principles and requirements for the safety and serviceability of structures, the basis of their design and verification that are given in EN 1990 – Basis of structural design. It is only concerned with the requirements for resistance, serviceability, durability and fire resistance of concrete structures.

Eurocode 8 applies to the design and construction of buildings and civil engineering works in seismic regions. Its purpose is to ensure, that in the event of earthquakes:

− human lives are protected,
− damage is limited,
− constructions after the earthquake remain operational.

EN 1998 – 3 contains provisions for the seismic strengthening and repair of existing buildings. Eurocode 8 Part 3 (EC8 Part 3) represents the only international document to address the issue of the analytical seismic assessment of buildings in a normative way. The decision to take such approach, a feature proper to the Eurocodes system, has entailed facing, in the phase of drafting the document, severe conceptual and practical challenges and it is anticipated that difficulties of various nature will be met in its application as well. Some of these difficulties will disappear with the future editions of the document, thanks to the progress made by the intense research activity currently devoted to the

subject. It would be vain, however, to expect that some time in the future the seismic assessment of an existing structure will become a matter of routine, similarly to what is possible for the design of a new one. The unavoidable limitation of knowledge, as much on the structural system as a whole, as on the single structural components, the difficulty in modeling behavior and capacity of components not intended to resist actions of seismic origin, the necessary use of less familiar and more complex (mostly non linear) methods of analysis and, perhaps most importantly, the measure of personal responsibility involved in the decisions the analyst has to take along the assessment process, are all elements that contribute into making any individual assessment a case of its own.

1.2.2 Concept of retrofitting

Retrofitting is the technical intervention in structural system of a building that improves the resistance to earthquake by optimizing the strength, ductility and earthquake loads. Strength of the building is generated from the structural dimensions, materials, shape, and number of structural elements. Ductility of the building is generated from good detailing, materials used, degree of seismic resistant, etc. Earthquake load is created from the site seismicity, mass of the structures, important of buildings, degree of seismic resistant, etc.

Due to the variety of structural condition of building it is hard to develop typical rules for retrofitting. Each building has different approaches depending on the structural deficiencies. Hence, engineers are needed to prepare and design the retrofitting approaches. In the design of them, the engineer must comply with the building codes. The results produced by the adopted retrofitting techniques must fulfill the minimum requirements on the buildings codes, such as deformation, detailing and strength.

1.2.3Performance based design

The seismic design of structures with levels of performativity (Performance-Based Design) is based on the principle of establishing an acceptable level damage (level performativity) depending on the probability of seismic vibration design, namely the determination of target seismic capacity. In other words, the method examines the actual way, which the construction will behave at various

power levels of seismic vibration design and corresponding expected level of damage. In this way, it is achieved an optimum combination of safety and economy.

1.3 Methods of analysis

According to the level of knowledge which achieved, in combination with the fulfillment certain, a requirement of regularity is determined the permissible method of analysis for construction of reinforced concrete. The analytical methods are provided.

- Linear elastic methods:
  - method of analysis with horizontal loads
  - Modal response spectrum analysis

- Non – linear methods:
  - Non – linear static analysis (pushover)
  - Non – linear time history analysis (dynamic)

The linear (first-order) elastic theory is traditionally used for analysis. With the aid of computer program, second-order analysis taking account of deflections in the structure can be performed. The maximum elastic load capacity is determined when any point in any member section reaches the yield stress or elastic critical buckling stress where stability is a problem.

Additionally constructions suffer significant inelastic deformation under a strong earthquake. Dynamic characteristics of the structure change with time so investigating the performance of it requires inelastic analytical procedures accounting for these features. The elastic analytical methods help to understand the actual behavior of structures by identifying failure modes and the potential for progressive collapse. Inelastic analysis processes basically include inelastic time history analysis and inelastic static analysis which is also known as pushover. Inelastic static analysis commonly referred to as "push over" analysis, shall be used to determine the reliable displacement capacities of a structure or frame as it reaches its limit of structural stability.

1.4 Objectives and Scope of the research

The objective of this thesis is the application and the comparison of various methods of elastic and inelastic analysis on a multi-storey reinforced concrete building and the proposal of a way to be reinforced. This study attempts to illustrate the differences of the analysis, the accuracy of the results as well as the comparison of the seismic response of the structure before and after the reinforcement. The study was implemented out on a 6-story building, which is located in Limassol, Cyprus. Analyses were carried out by using SAP 2000 V15.1 analysis program.

1.5 Organization of the thesis

The dissertation is organized in seven chapters:

In the *first chapter* of the thesis a brief description of the seismic activity in Cyprus and general situation of these days is made. Also the basic concepts of the retrofitting and for performance based design are mentioned .At the end a description of the analytical methods and eurocodes that were used is provided.

In the thesis' *second chapter* the studied structure is described, as well as the numerical model that was inserted at the analysis program SAP2000. Material characteristics, section properties and the way that the loads are transferred to the beams are offered.

After that (*third chapter)*, linear methods that were applied to the structure are analyzed. Firstly static analyses, modal analysis and modal response spectrum analysis based on EC8 methods are presented. Linear time history analysis was also used, by applying the accelerograms earthquake of Duzce in Turkey that took place in 1999. To compare the results of Linear Analysis Time history with Dynamic Spectral method, scale factors considered appropriate were used. Furthermore for each technique is shown analytically the procedure was followed in the program.

Subsequently, in the *fourth chapter,* the performance based frame design is described and the theoretical background is presented.

The *fifth chapter* describes the nonlinear static analysis approach, by illustrate the theoretical background. The results of pushover analysis are offered (push over curve, performance point, etc). Moreover, the process in the program for insert the parameters of the certain method, is displayed.

The *sixth chapter* of the thesis refers to the reinforcement methods of the existing structure. Various columns and beams are reinforced by concrete jackets and the retrofitted building's behavior

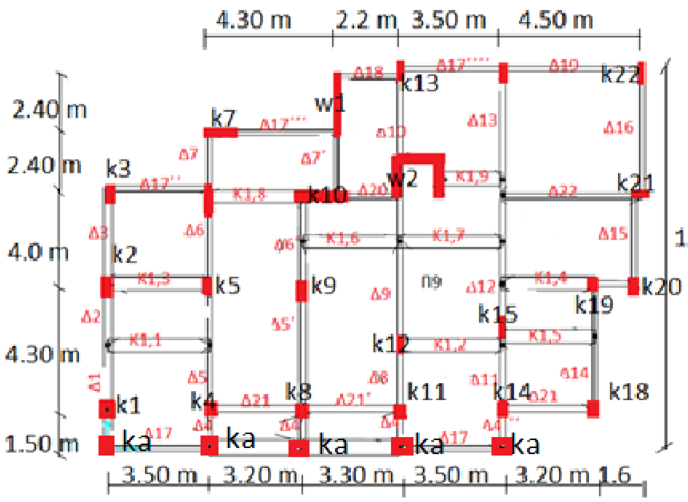
under seismic loading is compared to its original state. In addition it describes the non – linear time history analysis. Non – linear time history analysis was used, by applying the accelerograms earthquake of Northridge that took place in 1999. Furthermore the retrofitted building's behavior under the above seismic loading is compared to its original state.

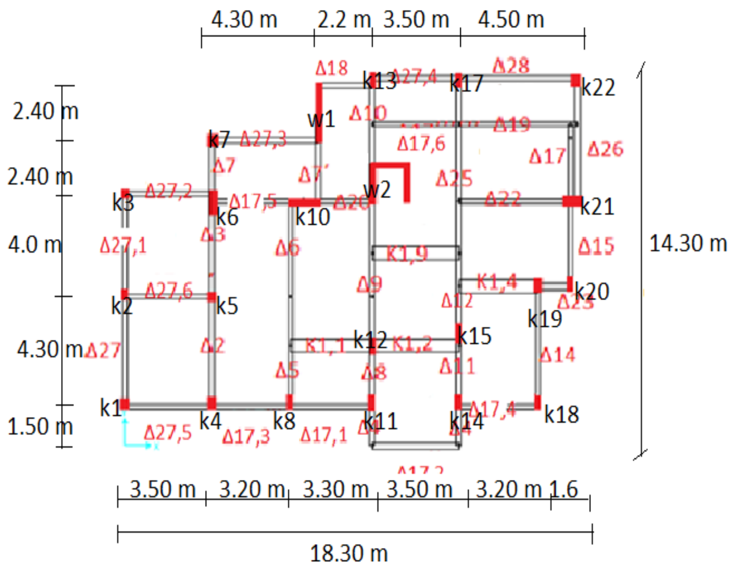
The last chapter (*seventh chapter)* of the thesis refers to the main conclusions from all of the above results which were obtained.

2. Description and simulation of existing building

2.1 Description of the structural system

The building studied in the current thesis is a six storey residential building in the area of Amathounta in the city of Limassol, which was built in 1982. It is constructed from reinforced concrete according to the early National Codes as has already been pointed out. The building consists of the ground floor, five storeys and the top floor. The main dimensions in plan are 18.30 meters in X direction and 14.30 meters in Y direction. The area of the sixth and the top floor of the building is reduced in size (because of the decrease in its length) compare to other floors' area. The four figures (1, 2, 3 and 4) below show the buildings plan view along their dimensions. The vertical support system of the building consists of columns and it has a relative symmetry, as shown in following plan views.





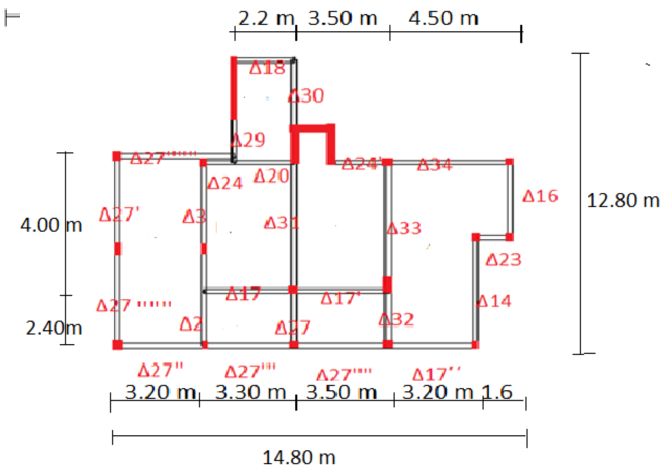


Figure 2.1: Plan view floor 1, 2, 3

Figure 2.2: Plan view floor 4

Figure 2.3: Plan view floor 5

The figure 5 below presents the plan view for the top floor in detail where the walls are founded.

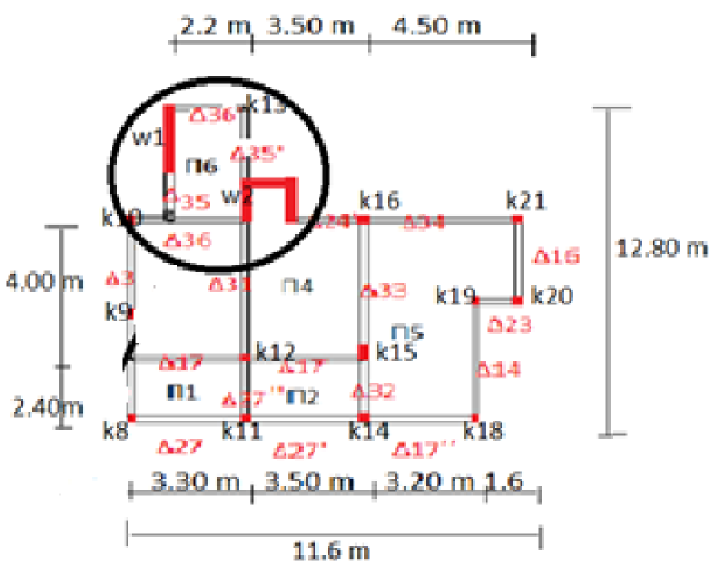


Figure 2.5: Plan view for the walls

Figure 2.4: Plan view top floor

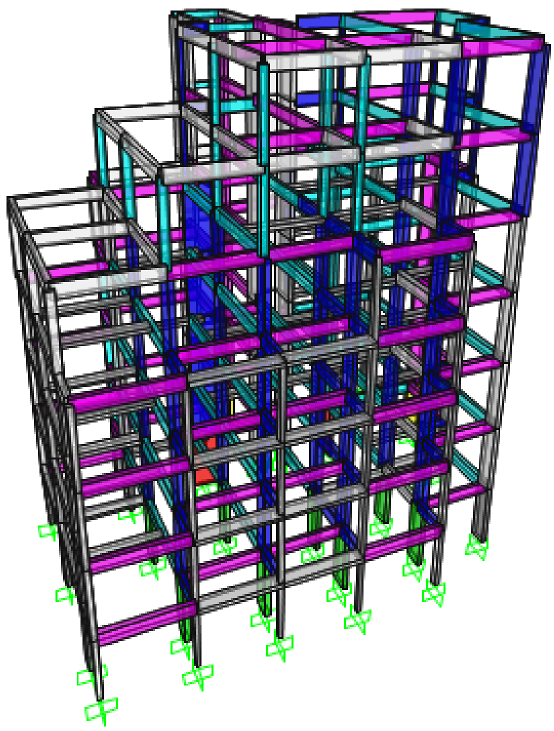


Figure 2.6 : Isometric view of the 3D model of the existing building

The total area of the building is 1528.57 m². The table below (table1) shows the total area and the area of each floor separately.

Number of storey	**Area (m²)**
ground floor	219.9
1	219.9
2	219.9
3	219.9
4	219.9
5	204.47
Top floor	125.42
Total Area	1528.57

Table 2.1: The area of each level

The building has seven levels over the ground. Each level has a standard height of 3.00 m while the height of the ground floor is 2.7 m. Figure 6 presents the section of the building at y direction along the height of each storey.

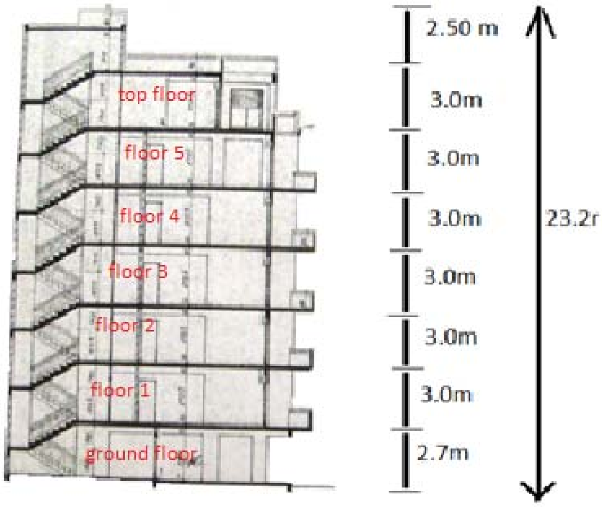


Figure2.7: Section A at y direction

2.1.1 Regularity in plan and regularity in elevation

The building satisfies the criteria imposed from EN1998 considering regularity in plan. With respect to the lateral stiffness and mass distribution, the structure is approximately symmetrical in plan with respect to the two orthogonal axes. The plane stiffness of the floors is sufficiently large in comparison to the lateral stiffness of the vertical structural elements which were considered as diaphragms. The slenderness of the building's plan view is provided from Eq.1 λ=Lmax/Lmin=14.3/11.60=1.3<4.0 (Eq.1) where Lmax and Lmin is the largest and smallest plan view dimension of the building respectively.

The structure does not satisfy the conditions for regularity in elevation because the last floor's dimensions –setback are smaller than 90% of the previous plan dimension as described from EN1998 [1]. However, the regularity in elevation is not considered to be valid , since the recess in the floor is not greater than 10% of previous dimension in plan view in the direction of recess (3.50 / 18.3 =19%> 10%) (Ec-8-1§ 4.2.3.3 (5) c).

2.2Materials

The bearing structure is made of reinforced concrete class C16/20. During the analysis the compressive strength of concrete is defined as fcm = fck +8 (MPa) = 22MPa.The specific weight of the reinforced concrete is 25 KN/m³.

In regard with the quality of the steel used in this study steel STAHL I with minimum limit of yield 2200kg/cm² and tensile resistance is 3400-5000kg/cm² as it is shown in Reinforced Concrete Regulations 1954. The measure elasticity of steel is taken as Es = 200GPa and the specific weight of steel as 78.5 KN/m³. The table (table 2) below shows the main attributes of the materials.

CONCRETE C16/20		**STEEL STAHL 1**	
Charactiristics		Charactiristics	
Fck (MPa)	16	E (GPa)	200
E (GPa)	28	Poisson's ratio (V)	0.3
Poisson's ratio (V)	0.2	minimun yield stress Fy (KN/m²)	215820
Fctm (MPa)	1.9	minimun tensile stress (Fu KN/m²)	392400
Specific weight (KN/m³)	25	Specific weight (KN/m³)	78.5

Table 2.2: The characteristic of the main materials

2.3 Description of section

The existing structure consists of reinforced concrete frames in both principal directions. The cross sections of beams and columns are categorized according to their dimensions in their area of reinforcement. Therefore, thirteen groups were created for beams and thirty groups for columns. The building which examined in this case study has four walls. These four walls are part of bearing structure, so they simulated in the model. The floor slabs are characterized by a thickness of 20 cm but they are not simulated in the model.

The fundamental characteristics of the structural elements (columns, beams and walls) are summarized as follows:

Columns: in the whole structure there are 159 reinforced concrete columns, collected in several groups depending on their dimensions and on their quantity of reinforcement. The figure below (figure 8)is a part of the original drawing of the column's design and it shows the geometry of the them and the number and type of the reinforcement. The dimensions and the reinforcement of almost each column vary along the floors. For example at the fifth floor column K1, K2, K3 and K7 reduce their dimensions..In addition these columns do not exist at the top floor.

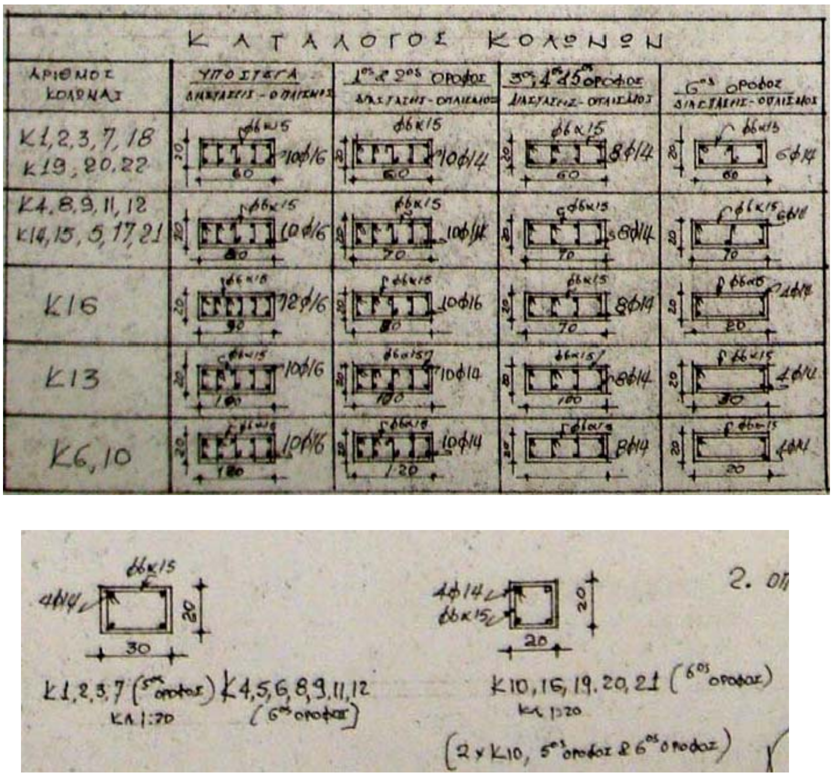


Figure 2. 8: Sketch of the original drawing of the column's design

<u>Beams:</u> The section of the beam usually follows the same pattern throughout the floors. There are different kinds of section but in the modeling it will be assumed that the beam section is rectangular. Also the beams are collected in several groups depending on their dimensions and on their quantity of reinforcement. In addition there are beams which are embedded in the plate (downsteam beams) and these are represented with symbol K. In this case study there are almost nine embedded beams in the plate, in each level. The next table (table 3) presents the groups of beam:

	REINFORCEMENT	**REINFORCEMENT**
<u>dimension (0.50 x 0.20)</u>	**TOP**	**BOTTOM**
beam 1	4 Y16	2Y16
beam 2,3,11,32	2Y16	4 Y16
beam 4	6Y16	2Y16
beam ,5 ,6 , 12, 14, 22 ,33	2Y12	5Y16
beam 7,10,15,17,20,21,28,29,30	2Y12	4Y12
beam 8	6Y16	3Y16

beam 9,13,16,19	2Y12	6Y16
beam 18,27	2Y12	3Y12
beam 23&24	2Y12	2Y12
beam 25 &26	2Y12	8Y16
beam 31	2Y12	7Y16
Dimension (0.30 x 0.20)		
beam 35	2Y12	4Y12
beam 36 & 37	2Y12	2Y12
dimension (0.5 x 0.15)		
Downsteam beam (name K 1)	10 Y 12 & 5 Y 6	
Downsteam beam (name K 2)	10 Y 12 & 5 Y 6	

Table 2.3: The groups of beam

Walls: In this case study has four walls which are part of bearing structure, so these walls are simulated in the model. The walls do not vary in section and reinforcement along the height and exist at all levels. The next table (table 4) presents the dimensions, reinforcement and the location of the walls:

Number of wall	Dimensions (m)	loaction x (m)	Location y (m)	Reinforcement
WALL1	2.30 x 0.2	7.8	12.85	Y14 @20
				Y6@15
WALL2	1.50 x 0.2	10	10.15	Y14 @20
				Y6@15
WALL3	1.50 x 0.2	11.5	10.15	Y14 @20
				Y6@15
WALL4	1.50 x 0.2	10.75	10.9	Y14 @20
				Y6@15

Table2. 4: The dimensions and the reinforcement for walls

2.4Loads

For this case is been used dead and live loads. The dead loads are the specific weight of the structure, the room finishing and the internal and external walls. The specific weight is depended on the materials which are selected. Concrete was used for this structure, so the specific weight is 25KN/m². The live loads of the structure are depended on the uses of the building.

The loads for the design of this building are shown in the following table (table 5).

	Deisign Loads	**Value**	
Permanent	self weight of reinforced concrete	25	KN/m^3
	internal walls	2.1	KN/m^2
	external walls	3.6	KN/m^2
	room finishing	2	KN/m^2
Live	rooms	2	KN/m^2
	balconies	4	KN/m^2
	stairs	3	KN/m^2

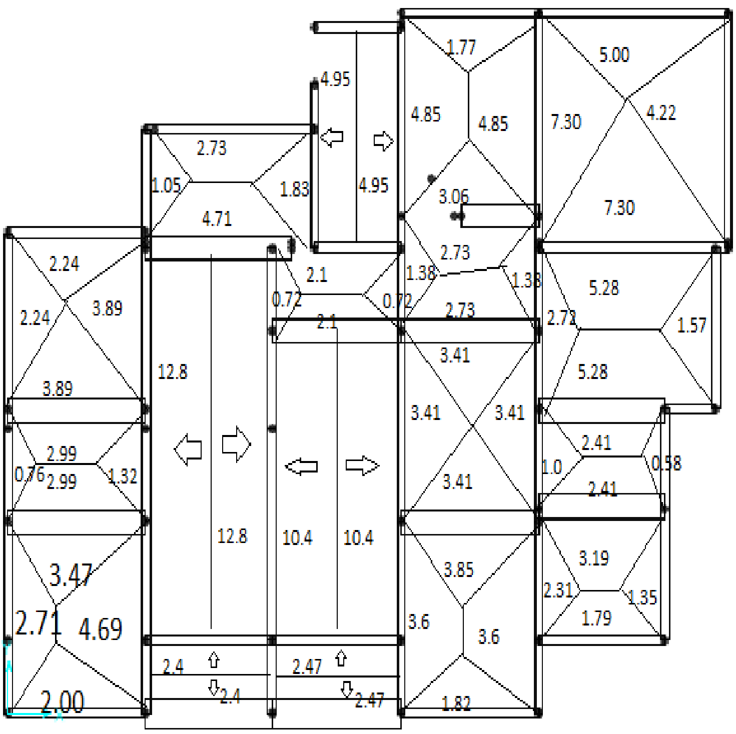
	roof	5	KN/m^2

Table 2.5: The dead and live loads

- The thick of the slab is 0.20 m ,so the distributed load of the slab due to the self weight of reinforced concrete is 25KN/m³* 0.20m = 5KN/m².
- The net height for all the walls is 2.5m .All internal beams have the same load from the internals walls, which is equal with 2.1KN/m²* 2.5m = 5.25 KN/m. All externals beams have load which is equal with 3.6KN/m²* 2.5m = 9.00 KN/m.

The slabs are not simulated in the model due to the fact that the load from specific weight and the roof finishing of them is transferred to the beams .The way that these surface loads are transferred to the beam is based on EKOS (§ 9.1.5). Specifically, when the two sides have the same support then the angle is 45 °.When the one side is pint support and the other is considered as fixed support then the angle to the side of the fixed support is 60°. In the same way the live loads are transferred from the slab to the beam.

The figures 9, 10, 11 and 12 below show the area of the slab which was multiplied by the distributed load to find the load of the beam.



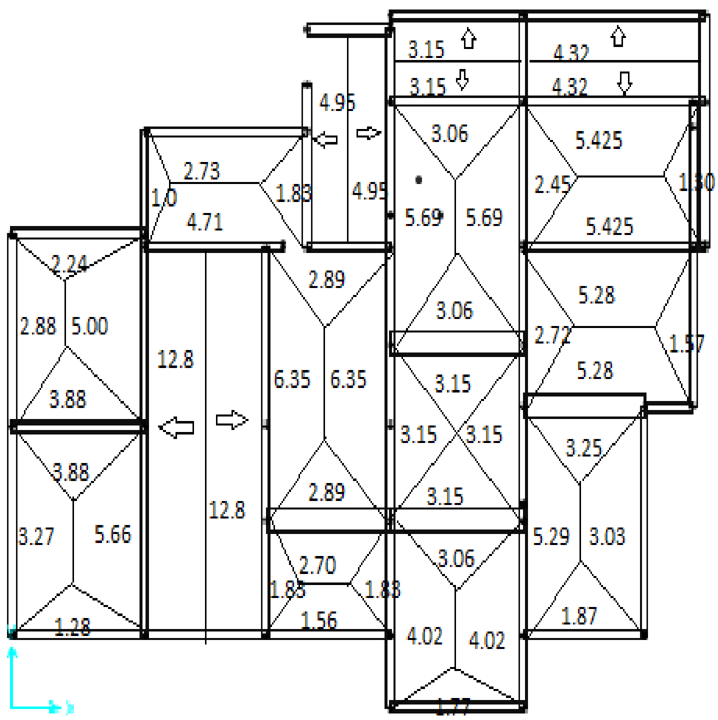


Figure 2.9: Loading surface for the beams of the floor 1, 2, 3   *Figure 2.10: Loading surface for the beams of the floor 4*

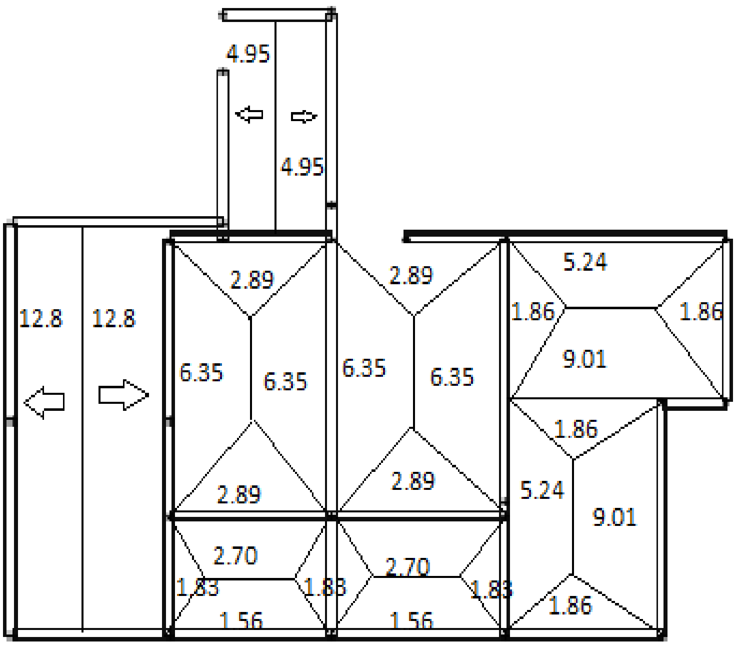


Figure 2.11: loading surface for the beams of the floor 5

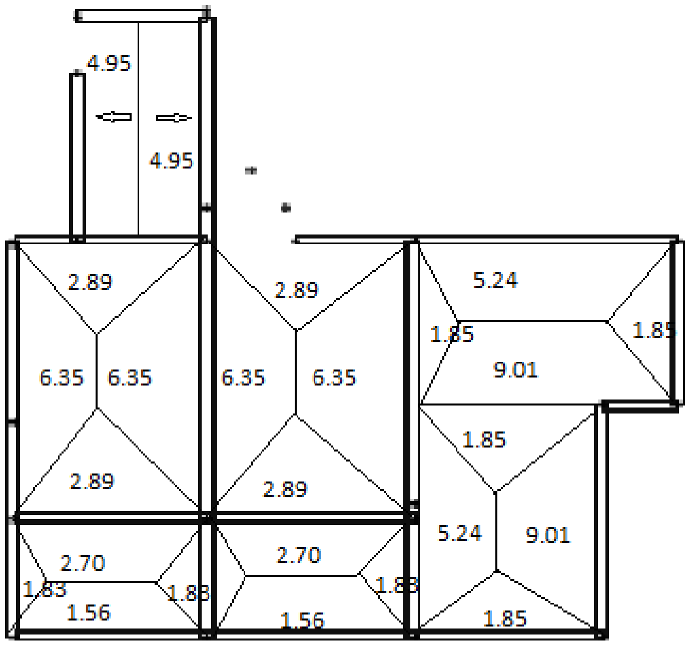


Figure 2.12: loading surface for the beams of the top floor

In the table below (table 6) is presented how the dead and live load were calculated. The loads were multiplied with the equivalent area and were divided by the length of the beam. As a result the load was distributed along the beam in KN/m. For example beam Δ1 (floor 1):

Dead load = (13.65 m²* 7 KN/ m²)/3.9m=4.86 KN/m

Live load = (13.65 m²* 2 KN/ m²)/3.9m=1.39 KN/m

Total dead load = dead load + load of external/internal wall

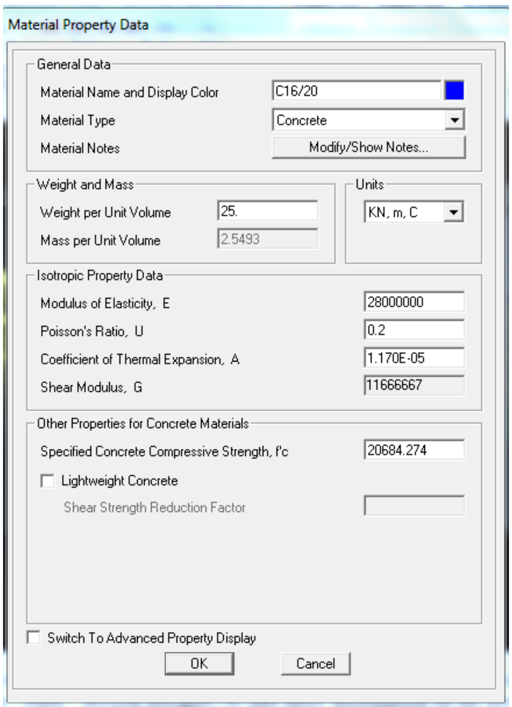
Total dead load =4.86 KN/m +9.00KN/m=13.86KN/m

NUMBER OF BEAM	AREA (m²)	LENGHT(m)	DEAD (KN/m)	wall KN/m	Total dead load	LIVE (KN/m)
Slab 1	13.65		7			2
Δ1	2.71	3.9	4.86	9	13.86	1.38
Δ4	4.69	3.9	8.41	5.25	13.66	2.4
Δ5	4.69	3.9	8.417	5.25	13.66	2.4
Δ17	2	3.5	4	9	13	1.14
K1.1	3.47	3.5	6.94	5.25	12.19	1.98

Table 2.6:Example for dead and live load for the beams Δ1, Δ4, Δ5, Δ17, k1.1.

2.5 Modelling

The first step of the simulation was to construct the 3D model of the existing building, considering the material properties and the element's geometry described before. The simulation of the building was done with the help of the program of analysis and dimensioning SAP2000 V15.0.0, through a context which consists of the contribution of vertical and horizontal linear elements and frame with six degrees of freedom (columns and beams, respectively). The columns and the beams were given as rectangular sections. Existing slab and walls were not introduced into the model, however, the impact load is taken into account by the appropriate linearly distributed load directly to the beams.



- The following procedure shows how to insert the material in the program :

Define → materials → Add new material

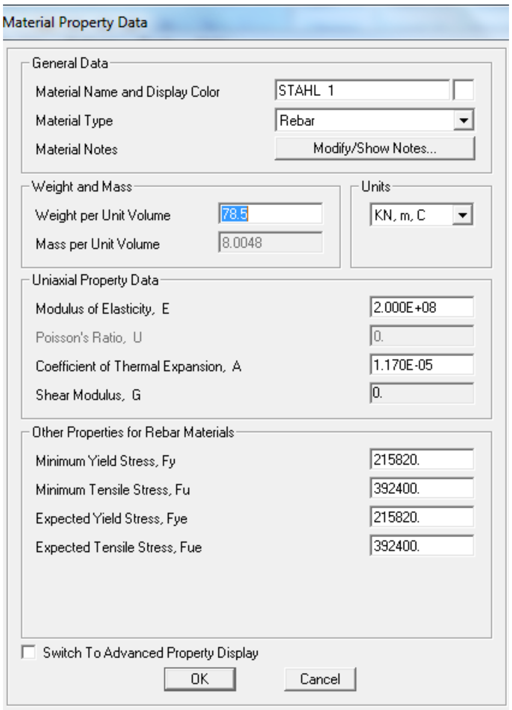
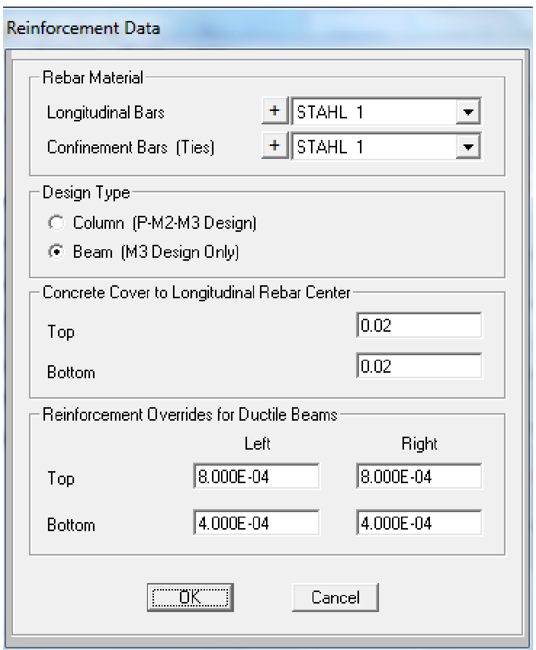


Figure 2.13: Insert materials to SAP 2000 (concrete and steel)

- Beams and columns can be inserted in the program with the command:

Define → Frame Sections → Add new property → Concrete and related options for beam or column. Alternatively, they may be designed through "section designer" (add new property - other - section designer) .This helps to take more information for the section (e.g. curvatures leakage and failure, moments drain) which is needed for the inelastic analysis to follow.

<u>For beams:</u>



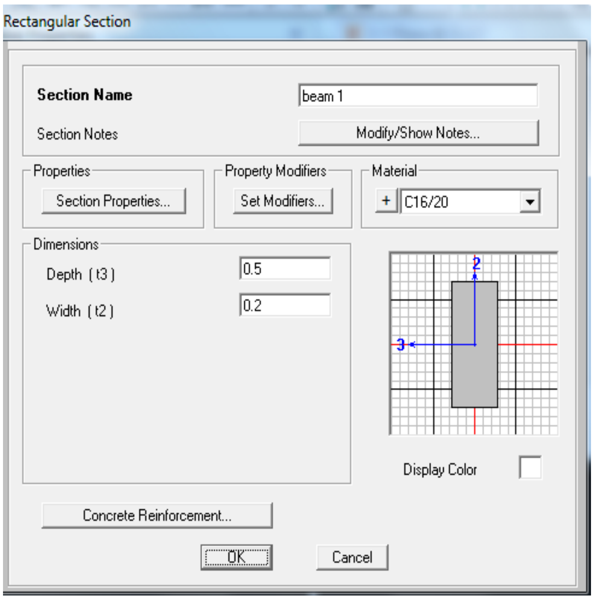
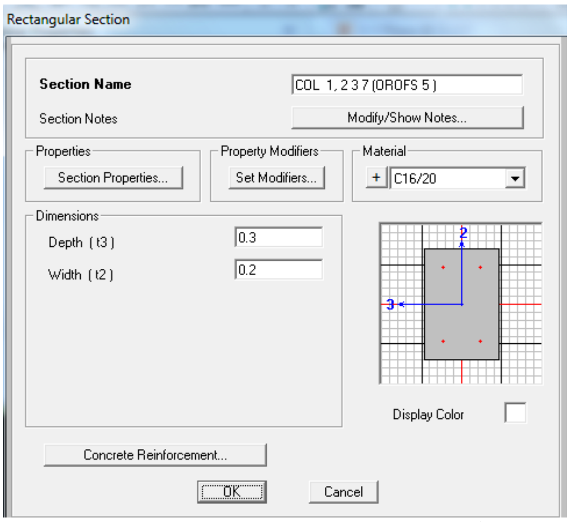


Figure2.14: Beam creation and import reinforcement

<u>For columns:</u>



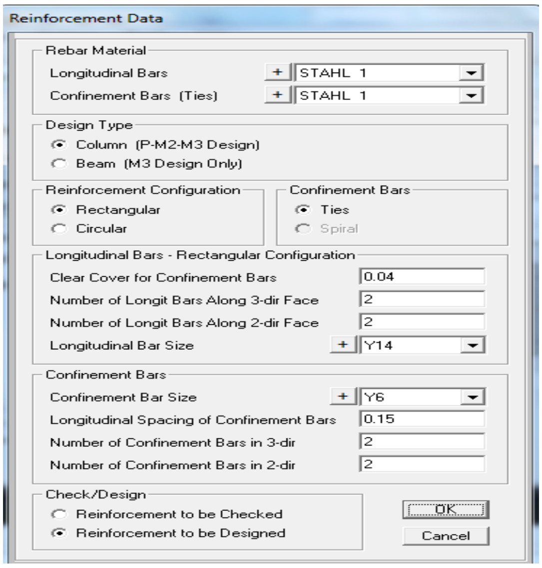
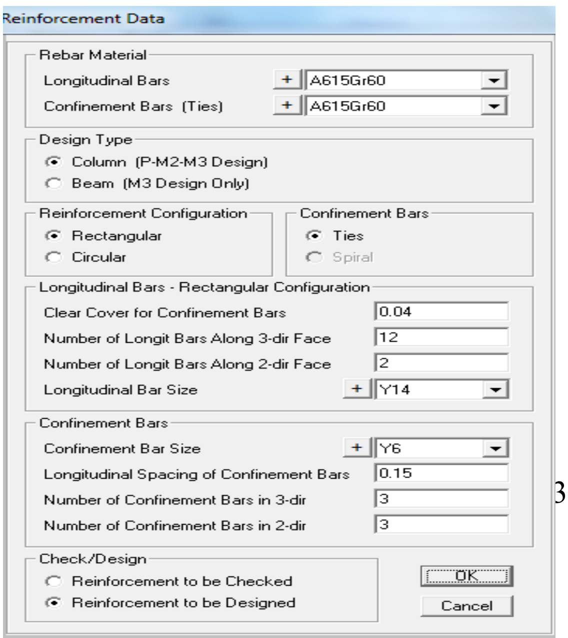


Figure 2.15: Column creation and import reinforcement

<u>For walls:</u>



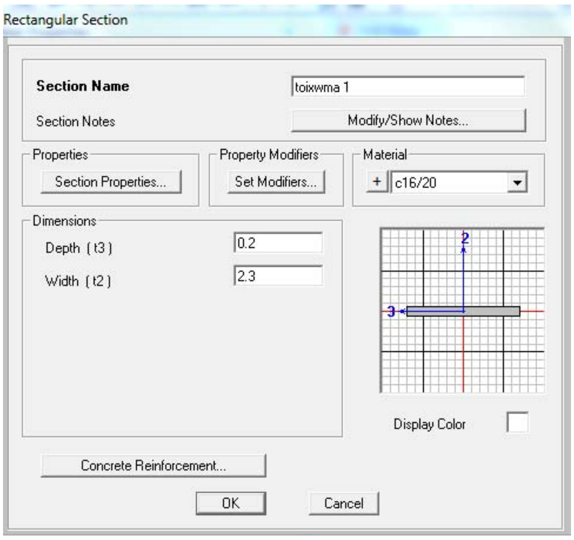


Figure2.16: Wall creation and import reinforcement

- The command for inserting the main grid is the following :

Define → Coordinates System/Grids → Global → Modify /show system

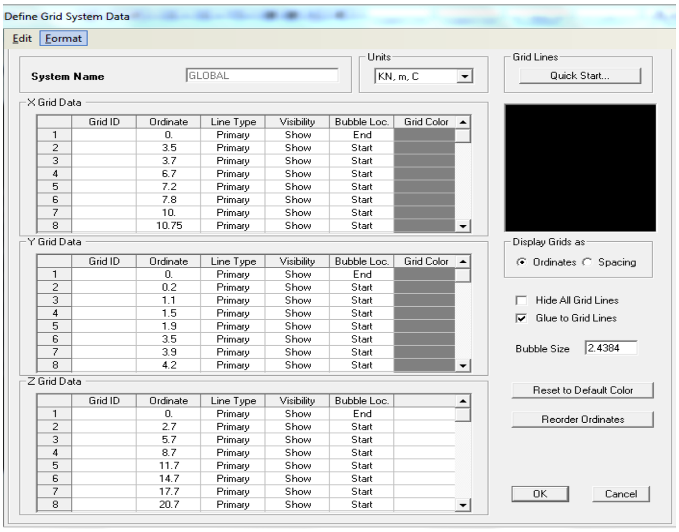


Figure 2.17: Insert the grid of the building

- In this model each base node is restrained in x, y, z, rx, ry, rx (fixed restraints), hence the soil-structure interaction is neglected. The procedure to introduce the support in the SAP2000 is presented below:

  Select the joints

  Assign → Joint → Restrains

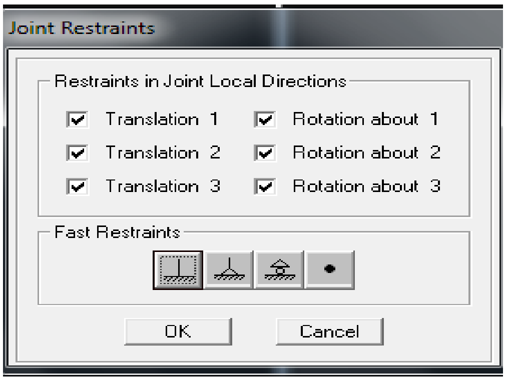


Figure 2.18: Insert the supports in SAP2000

- Beams are simulated from center to center of columns. In order for the beams not to bent some areas of must be considered as rigid elements (from the center until the end of the column).For example in the floor 4 were used nine different constrains. The procedure for the above is the following :

  Select the joint (usually two joints)

  Assign → Joint →Constrains → constrain type BOBY → add new constrain

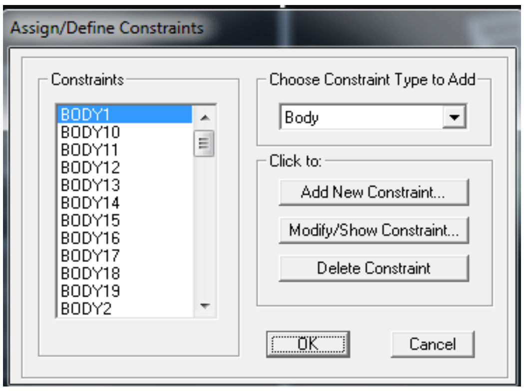


Figure 2.19: Insert body constrains in SAP2000

- It is a common practice that concrete floors in building structures, which typically have very high in-plane stiffness, are modeled with rigid diaphragm constraints (hereafter 'rigid diaphragms') for lateral load analysis. The diaphragm constraint creates links between joints, which are located within a plane such that they move together as a planar diaphragm, rigid against membrane (in-plane) deformation, but susceptible to plate (out-of-plane) deformation and associated effects. Diaphragm constraints relieve numerical accuracy problems which result when floor diaphragms are modeled with very high in-plane stiffness.

The diaphragm constraint was assigned separate at each level to simulate a rigid diaphragm. The command is the following:

- Select all elements at each level separate :

  Assign → Joint →Constrains → constrain type DIAPHRAGM → add new constrain

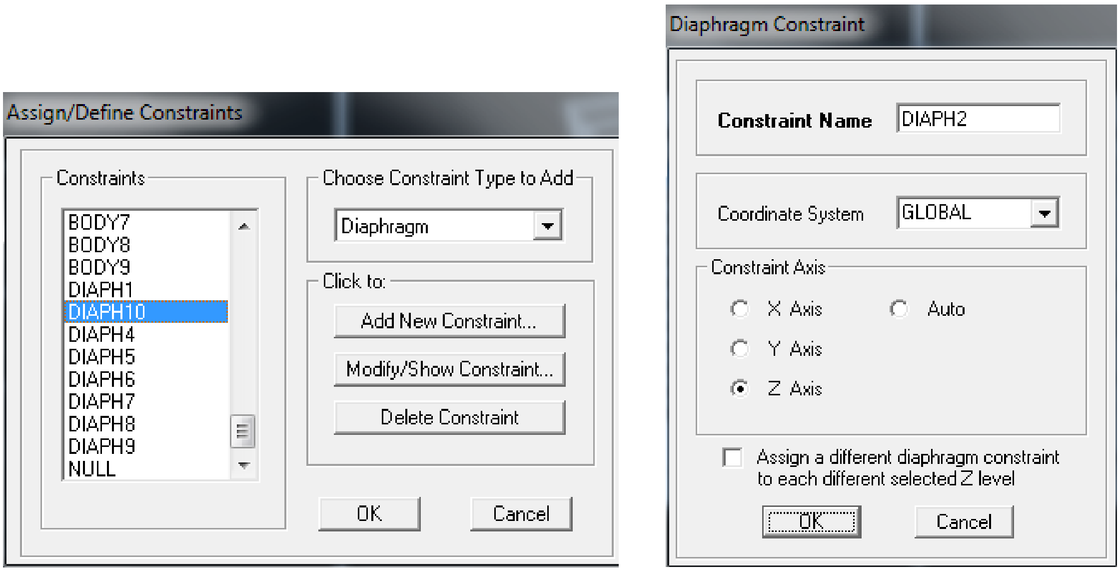


Figure 2.20: Insert diaphragm constrains in SAP2000

- Also the correction of the position of an element was done where was necessary.

This was achieved with the following commands :

Select the element

Assign → Frame →Insertion point → cardinal point 8) top center

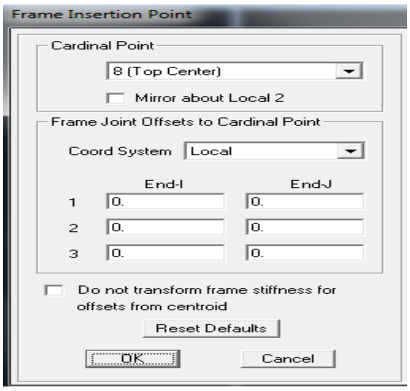


Figure 2.21: Insert offset in SAP2000

- The loads are inserted in the program with the commands below :

Select the element

Assign → Frame loads →Distributed

For live load must be created load pattern:

Define → Load patterns →add new load pattern

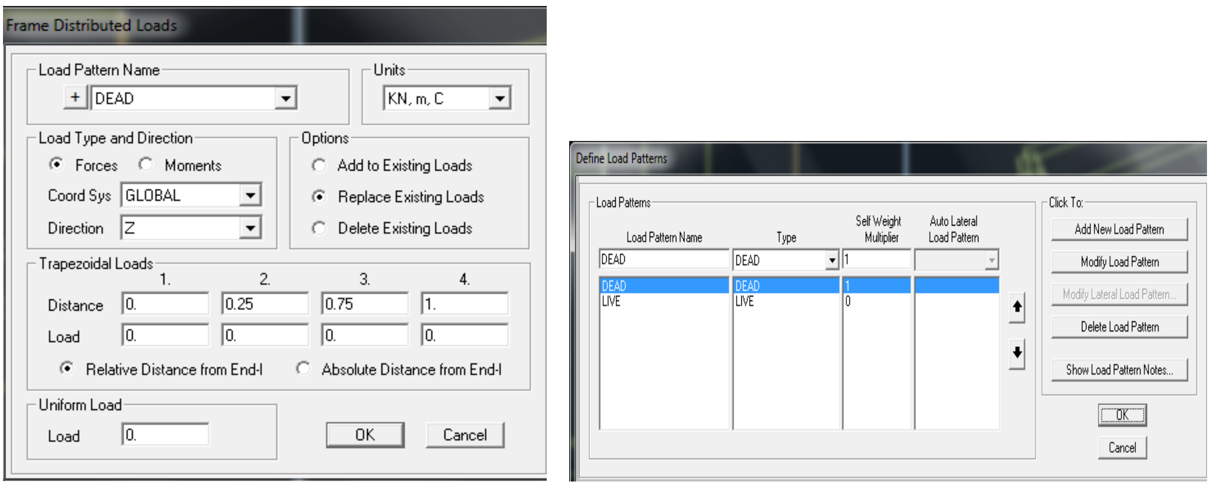


Figure 2.22: Insert dead and live loads in SAP2000

3. Elastic methods analysis

Introduction

The main objective of this thesis, as it was mentioned in the first chapter, is to perform the seismic assessment of the existing building. The seismic capacity is evaluated using pushover analysis. Pushover analysis is conducted using the numerical models, which were described in the chapter above. Anyway, before proceeding with this analysis, static, modal and response spectrum analysis were carried out. The results from each technique were compared in order to acquire the differences among the analysis methods.

The linear (first-order) elastic theory is traditionally used for analysis. With the aid of computer program, second-order analysis taking account of deflections in the structure can be performed. The maximum elastic load capacity is determined when a point in a member section reaches the yield stress. When the stability is a problem, because of the elastic critical buckling stress the maximum elastic load capacity is determined.

3.1 Static analysis

The load combinations of this analysis are two: the serviceability state design(SLS) and the ultimate state design(ULS). In the ultimate state design the usable load combination is: ULS = 1.35G + 1.5Q. For the serviceability state design the usable load combination is: SLS = 1.00G + 1.00Q. Below is going to be describing how these load combinations were calculated.

Ultimate limit state design: To satisfy the ultimate limit state design, the structure must not collapse when subjected to the peak design load. A structure is deemed to satisfy the ultimate limit state criteria when all factors (bending, shear and tensile or compressive stresses) are below the resistances, which were calculated for the current section. The basics load combinations of ULS are the follows:

- Basic combinations for permanent and transient design situations:

$$\sum_{j\ge1}\gamma_{G,j}G_{k,j}"+"\gamma_P P"+"\gamma_{Q,1}Q_{k,1}"+"\sum_{i>1}\gamma_{Q,i}\psi_{0,i}Q_{k,i}$$

$$\begin{cases}\sum_{j\ge1}\gamma_{G,j}G_{k,j}"+"\gamma_P P"+"\sum_{i\ge1}\gamma_{Q,i}\psi_{0,i}Q_{k,i} & \mathbf{0{,}85 \le \xi \le} \\ \sum_{j\ge1}\xi_j\gamma_{G,j}G_{k,j}"+"\gamma_P P"+"\gamma_{Q,1}Q_{k,1}"+"\sum_{i>1}\gamma_{Q,i}\psi_{0,i}Q_{k,i} & \end{cases}$$

- Basic combinations for seismic conditions design

$$\sum_{j\ge1}G_{k,j}"+"P"+"A_{Ed}"+"\sum_{i>1}\psi_{2,i}Q_{k,i}$$

Serviceability Limit State design: To satisfy the serviceability limit state criteria, a structure must remain functional for its intended use subject to routine loading, and as such the structure must not cause occupant under routine conditions. A structure is deemed to satisfy the serviceability limit state when the constituent elements do not deflect by more than certain limits, which are laid down in the building codes.

The basics load combinations of ULS are the follows:

- A typical matching (irreversible ULS)

$$\sum_{j\geq 1} G_{k,j} "+" P "+" Q_{k,1} "+" \sum_{i>1} \psi_{0,i} Q_{k,i}$$

- Partly completed-permanent combination (reversible ULS):

$$\sum_{i>1} G_{k,j} "+" P "+" \psi_{1,1} Q_{k,1} "+" \sum_{i>1} \psi_{2,i} Q_{k,i}$$

The table below presents the values of Ψ:

Action	ψ_0	ψ_1	ψ_2
Imposed loads in buildings, category (see EN 1991-1-1)			
Category A : domestic, residential areas	0,7	0,5	0,3
Category B : office areas	0,7	0,5	0,3
Category C : congregation areas	0,7	0,7	0,6
Category D : shopping areas	0,7	0,7	0,6
Category E : storage areas	1,0	0,9	0,8
Category F : traffic area, vehicle weight ≤ 30kN	0,7	0,7	0,6
Category G : traffic area, 30kN < vehicle weight ≤ 160kN	0,7	0,5	0,3
Category H : roofs	0	0	0
Snow loads on buildings (see EN 1991-1-3)*			
Finland, Iceland, Norway, Sweden	0,70	0,50	0,20
Remainder of CEN Member States, for sites located at altitude H > 1000 m a.s.l.	0,70	0,50	0,20
Remainder of CEN Member States, for sites located at altitude H ≤ 1000 m a.s.l.	0,50	0,20	0
Wind loads on buildings (see EN 1991-1-4)	0,6	0,2	0
Temperature (non-fire) in buildings (see EN 1991-1-5)	0,6	0,5	0
NOTE The ψ values may be set by the National annex. * For countries not mentioned below, see relevant local conditions.			

Table 3.1: The values of the coefficient Ψ

In the figures which are followed (Figures 3.1 & 3.2) it can be seen the deformed state of the building because of the two load combination ULS = 1.35G + 1.5Q and SLS = 1.00G + 1.00Q. Large deformations are observed in indirect supports.

ULS = 1.35 G + 1.5 Q

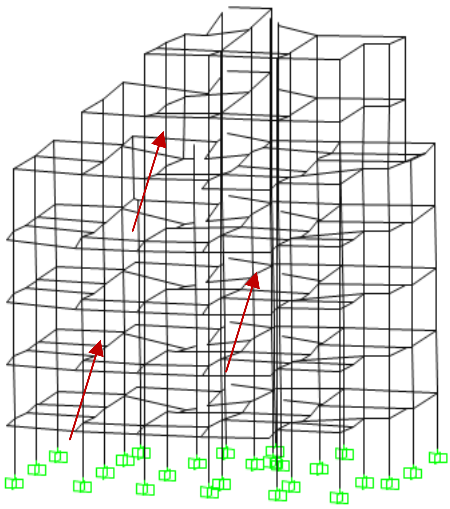


Figure 3.1: the deformed state of the structure for load combination ULS

- SLS = 1.00 G + 1.00 Q

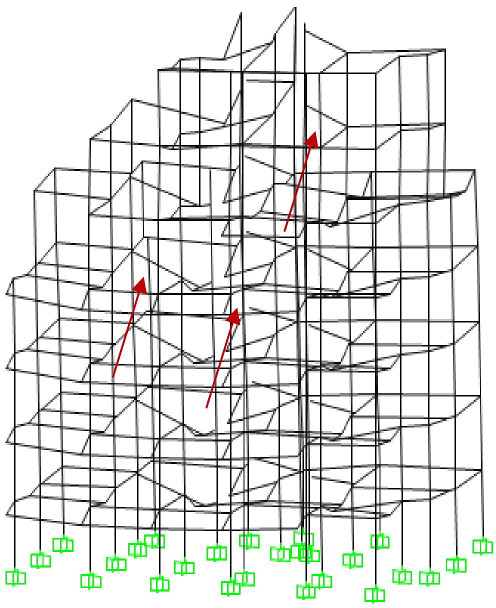


Figure 3.2: The deformed state of the structure for load combination SLS

3.2 Modal analysis

3.2.1 Theory of modal analysis

Eigenvalue analysis is a completely elastic structural analysis and it gives, as results, the modes of the structure. Particularly, natural periods, mode shapes and effective modal masses are obtained. Even if the frame models consist on 3D inelastic beam-column elements, the program is able to define the section's elastic properties directly, depending on the material type.

Modal analysis studies the dynamic properties or "structural characteristics" of a mechanical structure under dynamic excitation:

1. resonant frequency
2. mode shapes
3. damping

Modal analysis is the field of measuring and analysing the dynamic response of structures. Modal analysis uses the overall mass and stiffness of a structure to find the various periods at which it will naturally resonate. These periods of vibration are very important to note in earthquake engineering. It is imperative that a building's natural frequency does not match the frequency of expected earthquakes in the region in which the building is to be constructed. If a structure's natural frequency matches an earthquake's frequency, then the structure may continue to resonate and this brings structural damage. Modal analysis helps to understand how a structure vibrates (frequency, damping and mode shapes). Modal analysis can be used for:

- Troubleshooting
- Simulation and prediction
- Design optimization
- Diagnostics and health monitoring

3.2.2 Results for modal analysis

The combination of modal analysis is G +0.3 Q. The following table (Table 3.2) shows the values of the fundamental period of vibration of the building and the rates of participation of the masses (masses acting modal / total mass) for each mode and direction. It is observed that major modes for X and Y directions are the first and third respectively eigenmode .The rates of participation mass in percentage is MX = 31.8% and My=44.3 %.

Modal Participating Mass Ratios					
nber of m	Period (sec)	MX %	MY %	SMX %	SMY %
1	1.425	0.31623	0.00716	0.31623	0.00716
2	0.982434	0.23326	0.24204	0.54949	0.24919
3	0.814143	0.08745	0.44691	0.63694	0.6961
4	0.359218	0.11717	0.00615	0.75411	0.70225
5	0.295041	0.06774	0.07391	0.82185	0.77616
6	0.275324	0.07505	0.02778	0.89689	0.80394
7	0.206697	0.00111	0.00031	0.898	0.80425
8	0.166812	0.00183	0.00303	0.89984	0.80728
9	0.1603	4.18E-05	0.00195	0.89988	0.80923
10	0.14316	0.00022	0.01186	0.90009	0.82108
11	0.139469	0.00251	0.08952	0.90261	0.9106
12	0.127407	0.03874	0.00204	0.94135	0.91265
13	0.118713	0.00011	8.17E-07	0.94146	0.91265
14	0.113446	0.00027	0.00014	0.94173	0.91279
15	0.113094	0.00657	0.00155	0.9483	0.91434
16	0.111665	0.00154	0.00051	0.94984	0.91485
17	0.110501	0.00036	7.40E-05	0.95019	0.91492
18	0.105692	2.83E-05	5.06E-05	0.95022	0.91497
19	0.104972	0.00053	0.00038	0.95075	0.91535
20	0.103226	0.00123	0.00092	0.95198	0.91627
21	0.096522	0.00023	7.45E-06	0.95221	0.91628
22	0.094015	1.54E-05	4.41E-06	0.95223	0.91628
23	0.091369	0.00044	3.99E-05	0.95267	0.91632
24	0.088022	0.00045	0.00011	0.95312	0.91643

Table3.2: Modal Participating and Mass Ratios

The table 2 shows that the first Eigen mode T1 = 1.425 sec has very low rate of participation in the masses at y direction. Firstly the first mode is considered translational along the X-axis but it is primarily torsional. So it can be concluded that probably the building is torsionally sensitive. In the figures below (Figures 3.3, 3.4 & 3.5) it can be observed the first three modes of the modal analysis.

- First mode T1=Tx = 1.45 sec

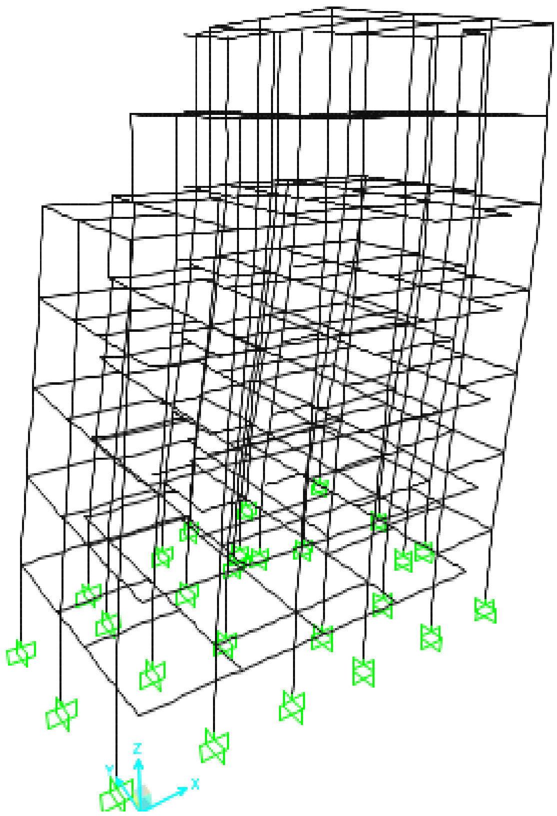


Figure 3.3: The deformed state of the first mode

- Second mode T2 =0.98 sec

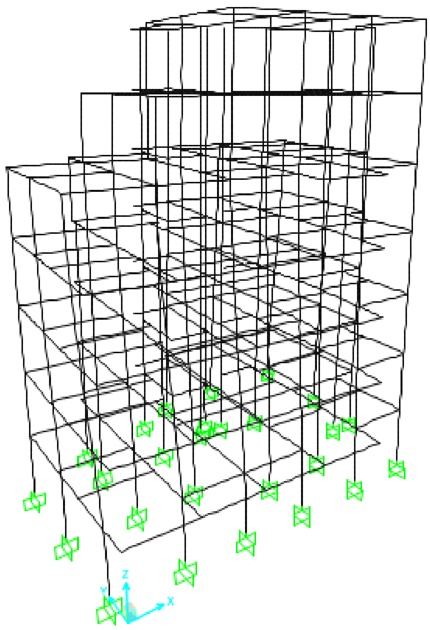


Figure3. 4: The deformed state of the second mode

- Third mode T3 = Ty=0.81 sec

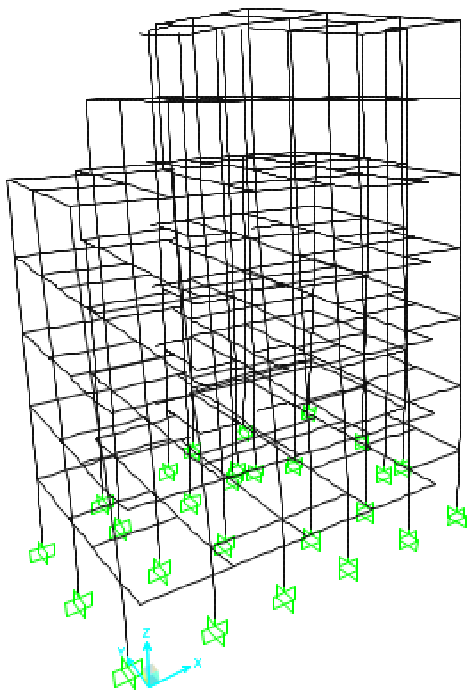


Figure 3. 5: the deformed state of the third mode

3.3Spectrum analysis

3.3 Response Spectrum Analysis

3.3.1 Theory of Response Spectrum analysis

Response spectrum analysis is a procedure for computing the statistical maximum response of a structure to a base excitation (or earthquake). Each of the vibration modes that are considered may be assumed to respond independently as a single-degree-of-freedom system. Design codes specify response spectra which determine the base acceleration applied to each mode according to its period.

Response Spectrum Analysis is used to determine peak displacements and member forces due to support accelerations. The "Complete Quadratic Combination" method (CQC) of combining modal responses is used to determine the peak response. This is equivalent to the "Square Root of the Sum of Squares" (SRSS) method if all modal damping ratios are zero.

Modal response spectrum analysis may be applied to all types of buildings without restrictions. Modes of vibration that contribute to the structure's global response are taken into account. Load combinations that are taken into account for seismic action are presented below:

G + 0.3 Q ± EEdx ± 0.30 EEdy

G + 0.3 Q ± EEdy ± 0.30 EEdx

where EEdx and EEdy represent the action effects due to the application of the seismic action along the axes x and y of the structure. The vertical component of the seismic action is ignored in this analysis.

There are advantages in using the response spectrum method of seismic analysis for prediction of displacements and member forces in structural systems. The method involves the calculation of only the maximum values of the displacements and member forces in each mode using smooth design spectra that are the average of several earthquake motions.

3.3.2 Response spectrum based on Eurocode 8(National Appendix Cyprus (CYS, 2005)

From January 1 is in force Eurocode 8 for seismic building design, while withdrawal of Cyprus Seismic Code in force since 1992 in Cyprus. Each Eurocode accompanied by the corresponding National Appendix which adjusts data Eurocodes tailored to each country of the European Union. So, in this chapter analyzed the National Appendix Cyprus (CYS, 2005) for Eurocode 8, making a parallel comparison of response spectrum in force under Cyprus Seismic Code and is presented above. As it can be seen in the new seismic map (Figure 3.6) of Cyprus, the seismic zones where reduced in three (compared to five in the seismic map of Cyprus in accordance with the Cyprus Seismic Code) and peak seismic ground acceleration have increased considerably.

Figure 3.6: Seismic zones of Cyprus in accordance with the National Appendix for Cyprus

The existing building is situated in zone 3 following the classification of the Cyprus Code. In this zone the expected peak ground acceleration ag in function of the gravity acceleration g is equal to 0.25g.

The table below shows the soil properties, the factor S and the periods T_B, T_C and T_D according to the National Appendix for Cyprus for EC 8.

Κατηγ. εδάφους	Περιγραφή	C_u (kPa)	S	T_B(s)	T_c(s)	T_D(s)
A	Βράχος	-	1.0	0.15	0.4	2.0
B	Αποθέσεις πολύ πυκνής άμμου, αμμοχάλικο	>250	1.2	0.15	0.5	2.0
C	Βαθιές αποθέσεις πυκνής ή	70-250	1.15	0.20	0.6	2.0

Table3. 3: Soil properties and the periods TB, TC and TD

The elastic response spectrum Se (T) for horizontal seismic forces, shown in figure below**.**

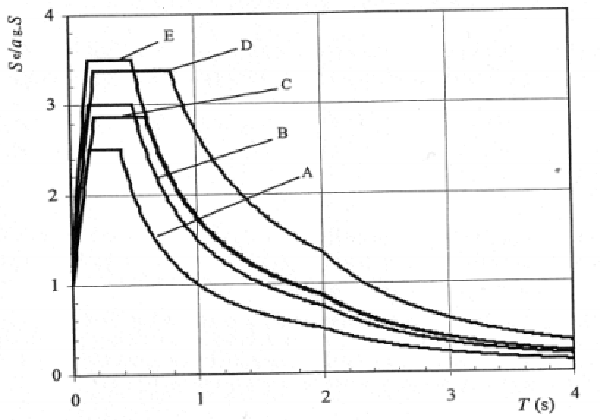


Figure3.7: Elastic response spectrum according to EC8

The following expressions where used for the application of this method:

$$0 \le T \le T_B: \quad S_e(T) = a_g \cdot S \cdot \left[1 + \frac{T}{T_B} \cdot (\eta \cdot 2{,}5 - 1)\right]$$

$$T_B \le T \le T_C: \quad S_e(T) = a_g \cdot S \cdot \eta \cdot 2{,}5$$

$$T_C \le T \le T_D: \quad S_e(T) = a_g \cdot S \cdot \eta \cdot 2{,}5\left[\frac{T_C}{T}\right]$$

$$T_D \le T \le 4\mathrm{s}: \quad S_e(T) = a_g \cdot S \cdot \eta \cdot 2{,}5\left[\frac{T_C T_D}{T^2}\right]$$

- Se (T), the elastic spectral acceleration

- T, the fundamental period of the linear system one degree of freedom
- ag, the ground acceleration on type A. The ground TB is smaller than the period at constant acceleration plateau spectrum
- TC is larger than the period at constant acceleration spectrum
- TD, the value that defines the beginning of the region of constant spectral displacement
- S, the soil factor: it is the correction factor depreciation value 1 for 5% viscous damping**.**

➢ The behavior factor for building made of concrete q=3.5

➢ The soil category is B so the values are the following:
  - S=1.2, η=1 and characteristic periods are TB=0.15sec, TC=0.5 sec, TD=2.0.

➢ The building is located in Limassol so the seismic zone is the third. Therefore the ground acceleration is αg = 0.25g.

➢ The rate lower limit is β=0.2

The elastic response spectrum in terms of accelerations is constructed following **(**Figure 3.8) the relationships described in above. The spectrum which is used is the following:

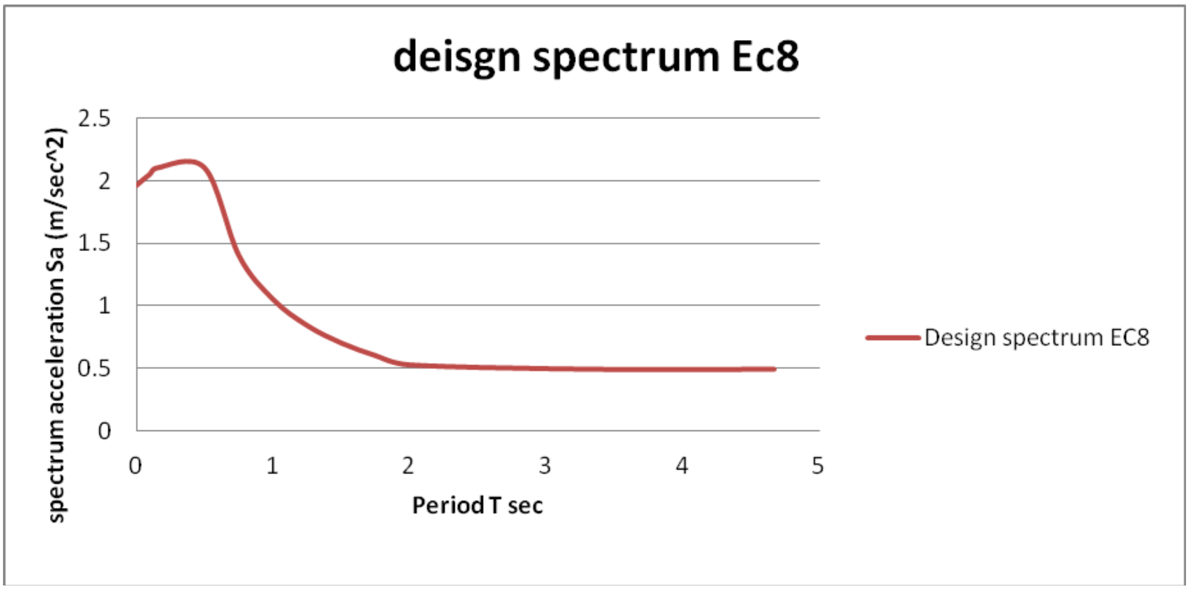


Figure.3.8: Elastic response spectrum which is used

3.3.3Results of response spectrum analysis

Load combinations that are taken into account for seismic action are presented below.

Quake in direction x:

$G + 0.3\ Q + E_x + 0.30\ E_y$

$G + 0.3\ Q + E_x - 0.30\ E_y$

$G + 0.3\ Q - E_x + 0.30\ E_y$

$G + 0.3\ Q - E_x - 0.30\ E_y$

Quake in direction y:

$G + 0.3\ Q + E_y + 0.30\ E_x$

$G + 0.3\ Q + E_y - 0.30\ E_x$

$G + 0.3\ Q - E_y + 0.30\ E_x$

$G + 0.3\ Q - E_y - 0.30\ E_x$

The vertical component of the seismic action is ignored in this analysis. The table below illustrates the maximum displacements of the structure derived from the modal response spectrum analysis.

load combination :	linear add	Max Displacements				
joint displacement	U1 (m)	U2 (m)	U3 (m)	R1 (radians)	R2(radians)	R3(radians)
ground floor	0.004615	-0.0041	-0.0047	0.002876	0.003238	0.000364
floor 1	0.014388	-0.0129	-0.0061	0.003462	0.003561	0.001121
floor 2	0.024408	-0.0225	-0.0067	0.003454	0.0035	0.001873
floor 3	0.033042	-0.031	-0.0069	0.003375	0.003475	0.002485
floor 4	0.03912	-0.0382	-0.006	0.00223	0.001998	0.002923
floor 5	0.037728	-0.0372	-0.0054	0.002107	0.001334	0.002923
top floor	0.039139	-0.042	-0.0057	0.002088	-0.001298	0.002938

Table3 4: Maximum displacements according to the modal response spectrum analysis

In the followed table it can be observed the maximum values for the axial force, the shear force and the moment for the base column.

load combination :	**linear add**	**Max Columns' Forces**			
	P (KN)	**V2 (KN)**	**V3(KN)**	**M2(KNm)**	**M3(KNm)**
ground floor	-2058.41	-210.641	-487.491	-3217.67	-912
floor 1	-1788.16	-126.568	477.32	-1901.33	-384.78
floor 2	-1504	-108.86	319.27	-808.54	246.72
floor 3	-1218.69	-94.63	202.06	-648.76	295.368
floor 4	-935.56	98.45	366.24	-1300.94	295.37
floor 5	-659.13	31.01	-559.079	-1300.93	-54.9
top floor	-180.76	-48.71	152.55	-410.55	-71.82

Table3. 5: Maximum forces of the column according to the modal response spectrum analysis

The element that receives the greater axial force is the wall 1. It is located in the ground floor. The column K10 has the maximum shear force (V2) and moment (M3). The maximum M2 and V3 are in the wall 2 and wall 3 respectively. The diagrams of the shear force V2 and the moment M3 in the column K10 with load combination G+0.3Q+EX-0.3EY are presented in the diagrams beneath.

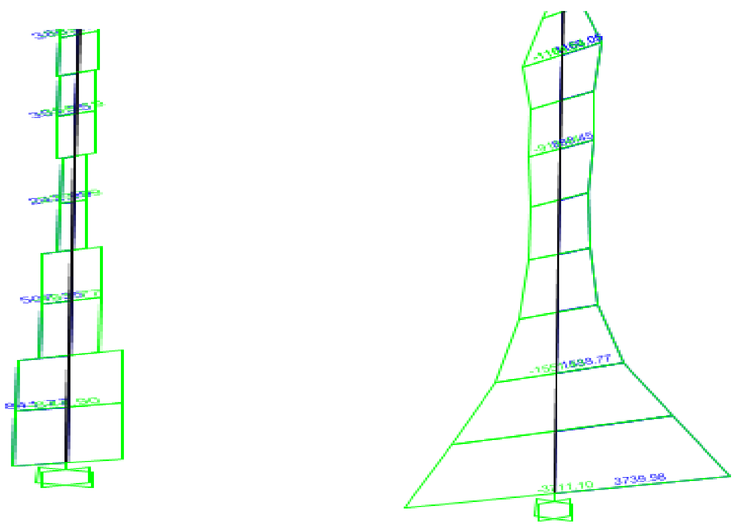


Figure 3.9: The diagrams of the shear force and moment for the column K10

3.4 Modelling

3.4.1 Load combination ULS and SLS

Two load combinations (ultimate and serviceability) were created with the commands below:

- ULS = 1.35 G + 1.5 Q
- SLS = 1.00 G + 1.00 Q.

Define →Load combination → Add new combo

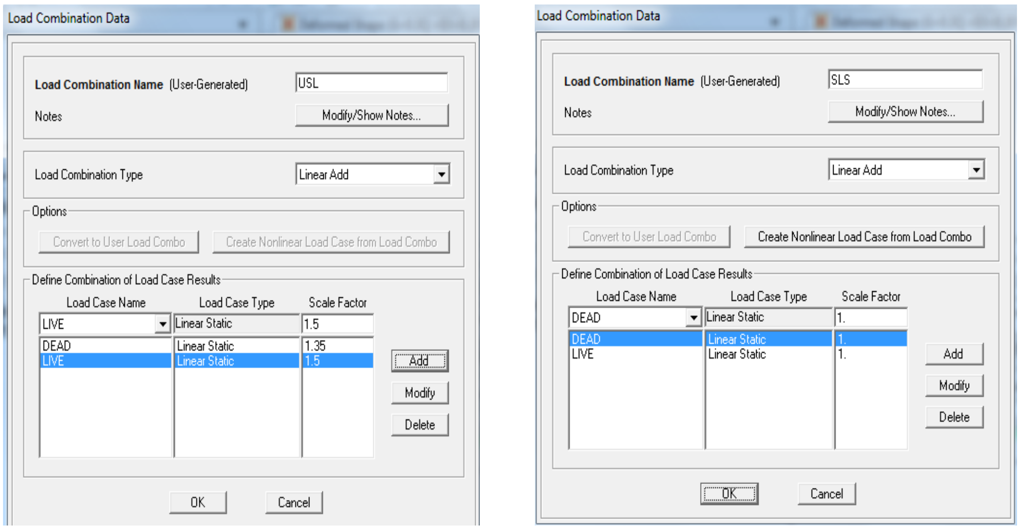


Figure 3.10: Insert load combination ULS and SLS in SAP2000

3.4.2Modal analysis

The procedure for modal analysis (G +0.3 Q) is the following:

Define →Mass source → mass definition (from loads)

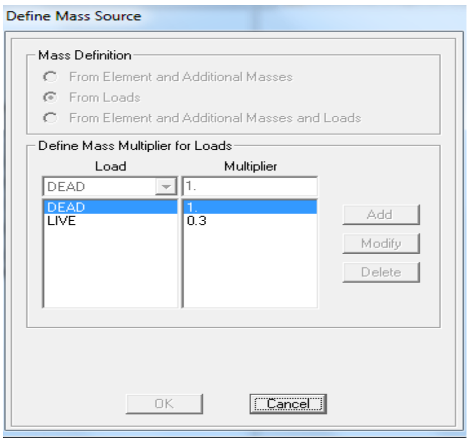


Figure 3.11: The mass definition modal analysis

3.4.3 Response spectrum analysis

The design spectrum was introduced in Sap2000 by the succeeded procedure:

Define →Function → Response Spectrum →Function type (Eurocode 2004) → Show / modify spectrum

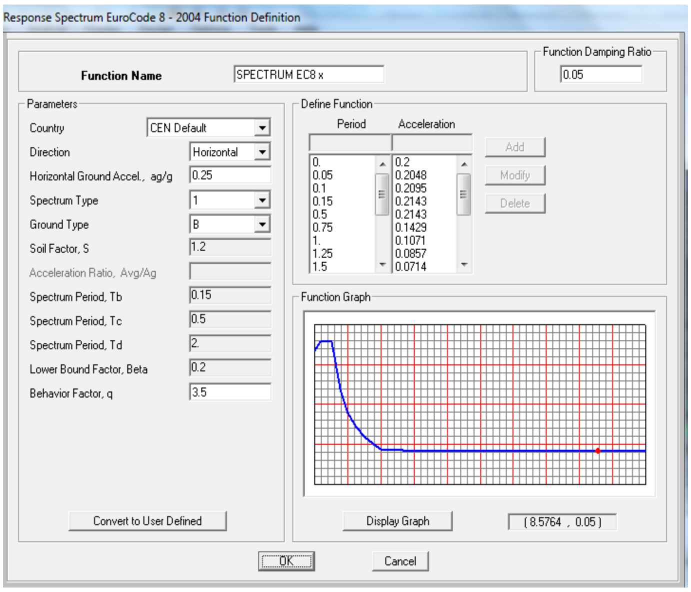


Figure 3.12: Inserting the design spectrum according to Eurocode 8

The load categories were defined corresponding to the design spectrum on each X and Y direction. The procedure is presented below:

Define →Load cases → Add new load case

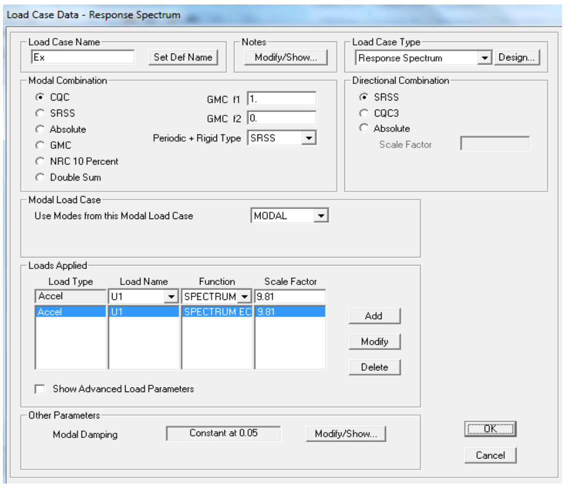


Figure 3.13: The load cases were defined for the design spectrum

The procedure for inserting the load combinations that are taken into account of the seismic action are presented below:

$$G + 0.3\,Q \pm EEdx \pm 0.30\,EEdy$$

$$G + 0.3\,Q \pm EEdy \pm 0.30\,EEdx$$

The procedure remains the same for direction Y.

- Define →Load combination → Add new combo → EEdx ± 0.30 EEdy (with combination type SRSS)

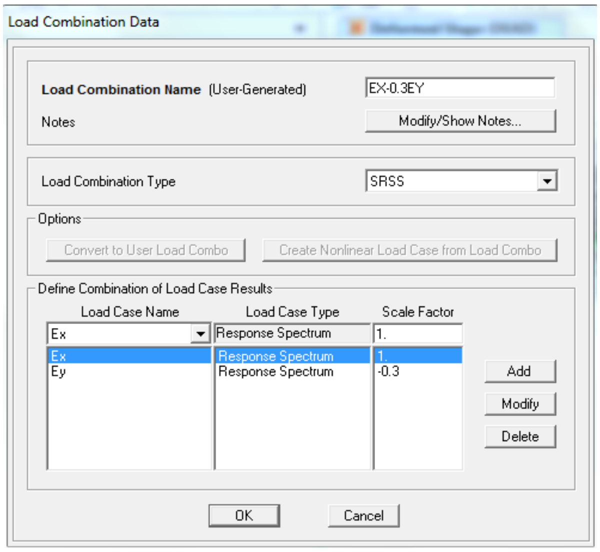


Figure 3.14: Combination of seismic loading in X Direction

- Define →Load combination → Add new combo → G + 0.3 Q ± EEdx ± 0.30 EEdy (With combination type Linear add)

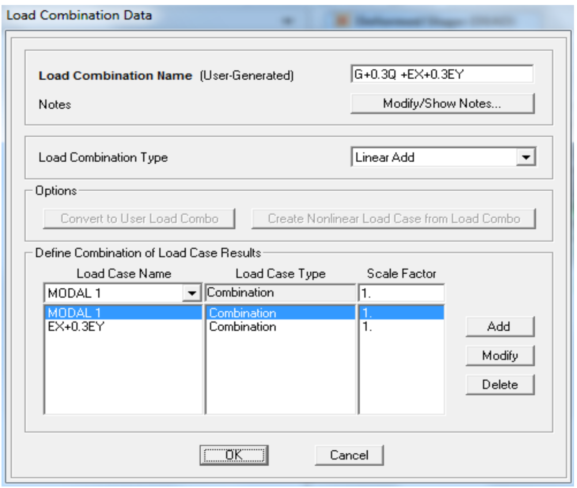


Figure 3.15: Combination MODAL + EX +0.3 EY in X Direction

3.5 Linear time history analysis

Time history dynamic analysis of structures is time consuming for problems with lots of number of degrees of freedom. Time-history analysis is provided for linear or nonlinear evaluation of dynamic structural response under loading which may vary according to the specified time function. In linear time history analysis for elastic analysis of the structure applied seismic loading, which is expressed by soil vibration and accelerograms performed solving the dynamic problem for every time. The resulting response is very sensitive to the basic changes system parameters (agitation, mass, stiffness, damping).

3.5.1 Data for time history analysis

The time history analysis examines the response of the structure when the positive charge in the three directions x, y, z is given as accelerograms. In the present case study, it was used by applying the accelerograms earthquake of Duzce in Turkey at 1999 record P1540 in the directions x and y. The accelerograms that used comes from measurements made at the surface of the station in Duzce.

Initially, the data of the earthquake were introduced in the SeismoSignal to get the accelerograms. The earthquake recordings are at steps of 0.005 sec, its total length is 25.870 sec and the time steps are 5177. The maximum acceleration was 0.357g at direction Y and 0.535g at direction X. The magnitude was M=7.1 and the modal damping is equal to 5%.

Below are presented the earthquake accelerograms and the response spectrum in Turkey at the two directions as they appear in the program.

- Y direction:

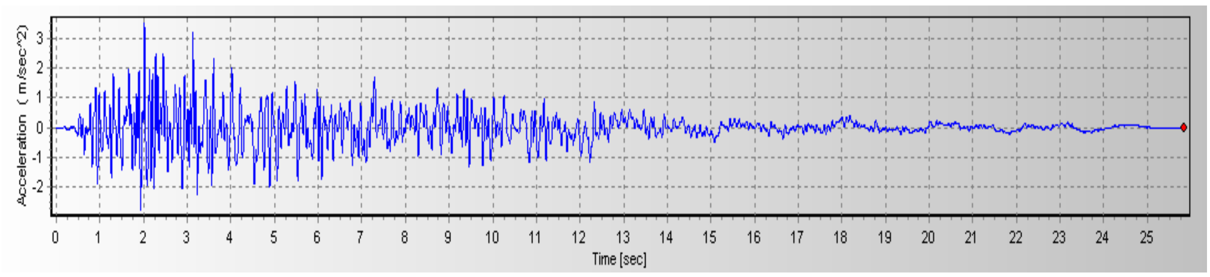


Figure3.16: Accelerogram for earthquake Duzce at Y direction

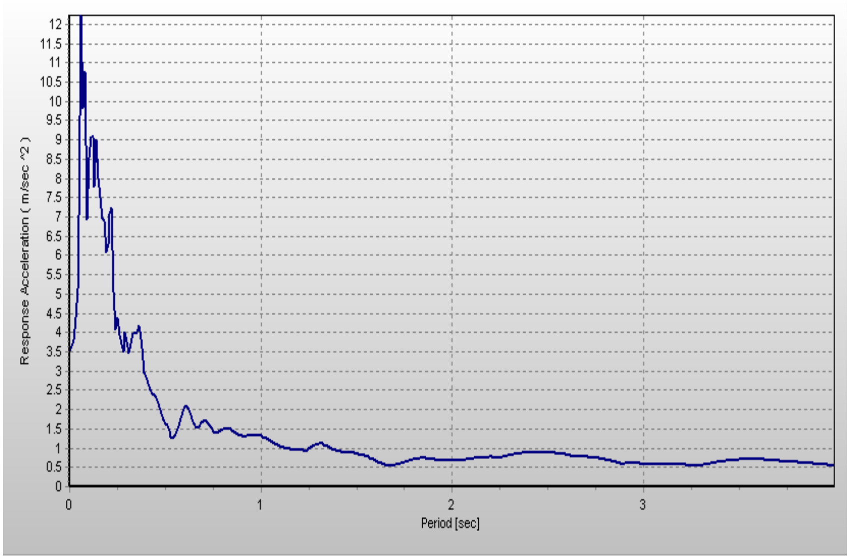


Figure3.17: Response spectrum for earthquake Duzce at Y direction

- <u>X direction :</u>

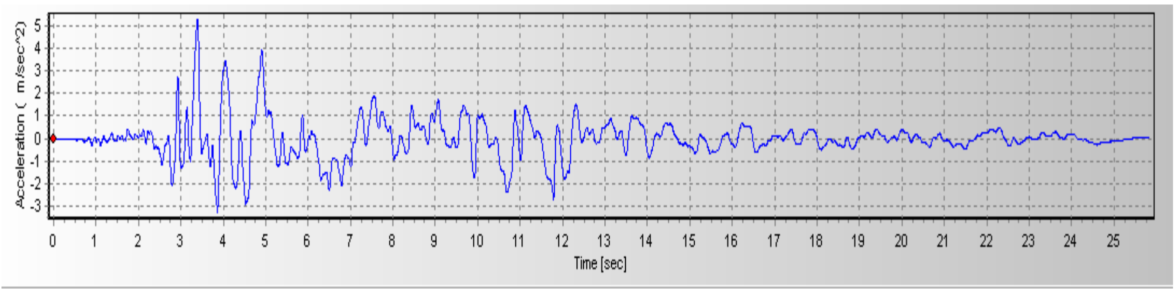


Figure3.18: Accelerogram for earthquake Duzce at X direction

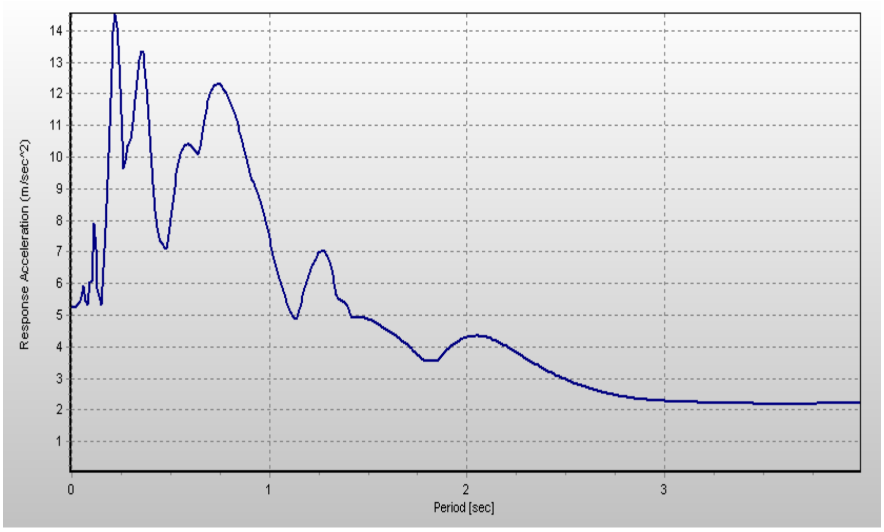


Figure3.19: Response spectrum for earthquake Duzce at X direction

3.5.2 Results for time history analysis

The previous accelerograms were introduced in SAP2000 for two different directions of X and Y (U1 and U2 respectively). The modal damping is equal to 5%. The load combination was used G+ 0.3Q + EX + EY. For two components of the earthquake X and Y are become spatial superposition SRSS and then adds with the linear superposition the G+ 0.3 Q. The results of the analysis are shown below. Table below shows the maximum displacements of the structure derived from the time history analysis.

load combination :		**Max Displacements**				
joint displacement	**U1 (m)**	**U2 (m)**	**U3 (m)**	**R1 (radians)**	**R2(radians)**	**R3(radians)**
ground floor	0.026058	0.017621	-0.010324	0.009642	0.01244	0.001637
floor 1	0.075245	-0.055199	-0.017918	0.001102	0.01372	0.005824
floor 2	0.127904	-0.092135	-0.016745	0.00997	0.012936	0.009437
floor 3	0.174771	-0.121976	-0.015058	0.008858	0.015108	-0.001173
floor 4	0.208395	-0.144778	-0.013984	0.006705	0.009409	0.013405
floor 5	0.193224	-0.137767	-0.007844	0.005807	-0.003685	0.013405
top floor	0.200254	-0.152308	-0.00828	0.005722	0.005514	-0.013488

Table 3.6: Maximum Displacements according to time history analysis

The Table 3.7 demonstrates the maximum values of the axial force, the shear force and the moment for the base column.

load combination :	**linear add**	**Columns' Forces**			
floors	**P (KN)**	**V2 (KN)**	**V3(KN)**	**M2(KNm)**	**M3(KNm)**
ground floor	-4070.25	879.13	-1655.2	-8990.59	-3882.39
floor 1	-3794.59	-537.44	1388.68	-5207.69	-1596.7
floor 2	-3456.25	-367.15	889.16	-2871.26	-1393.55
floor 3	-3057.68	-519.987	684.02	-2625.28	1.330
floor 4	-2641.02	396.79	1889.76	-5523.59	1190.38
floor 5	-2359.43	151.76	-2.829	-5523.59	-274.41
top floor	-207.18	342.1	989.46	2968.38	555.43

Table 3.7: Maximum forces of the columns according to the time history analysis

3.5.2.1 Time profile for displacement and forces

In this section dynamic response of the internal stress for some elements will be presented. The time course of the internal stress according to the accelerograms delivers the most effective results in the time history analysis. In the follow figures (Figure 3.20 & 3.21) it can be seen the time course of the axial force for the wall 1. The wall is located in the ground floor. The figures are for the horizontal and vertical component of the seismic action in directions X (Duzce X – X) and Y (Duzce Y-Y) respectively.

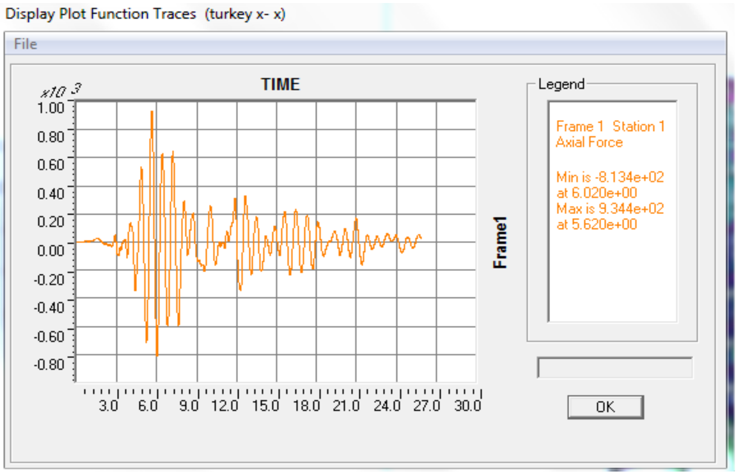


Figure 3.20: The diagram of the variation of the axial force for the wall1 during Duzce-Turkey x- x .

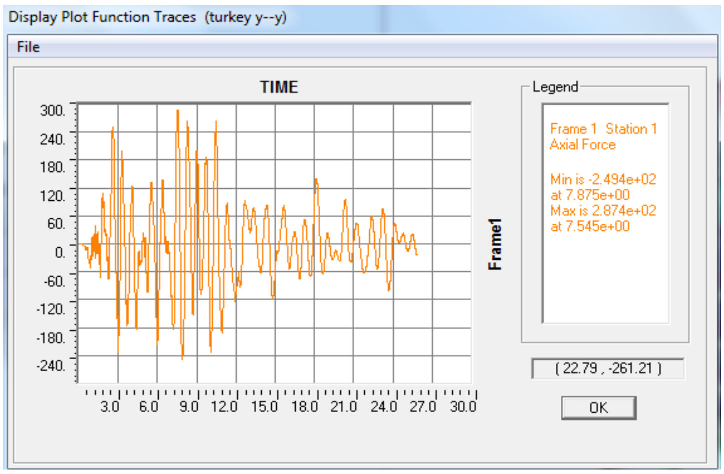


Figure 3.21: The diagram of the variation of the axial force for the wall1 during Duzce-Turkey Y-Y .

The absolute value of the maximum axial force:

Turkey X-X: 922 kN at 5.85 sec

Turkey Y- Y: 274 kN at7.05 sec

The figure underneath shows the time responses of the shear force V2 and the moment M3 for the column K10 (it is located in the ground floor). The red and the green color illustrate the moment M3 and the shear force V2 respectively, in relation with time.

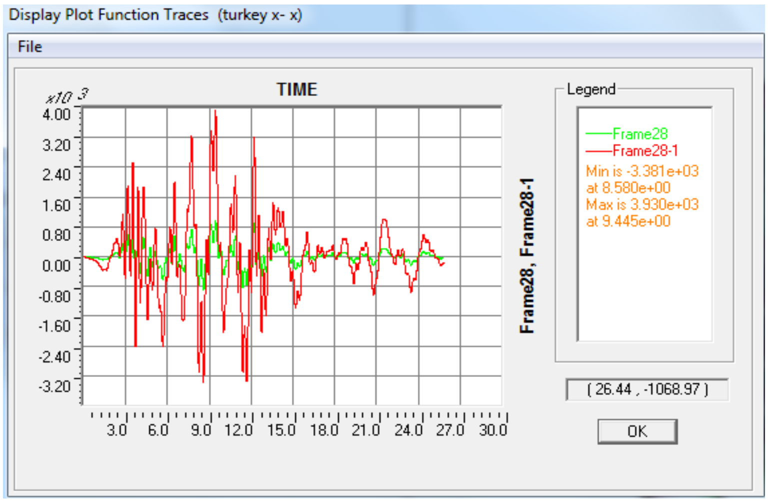


Figure 3.22: The diagram of the variation of the shear force V2 and moment M3 for K10 during Duzce-Turkey x- x .

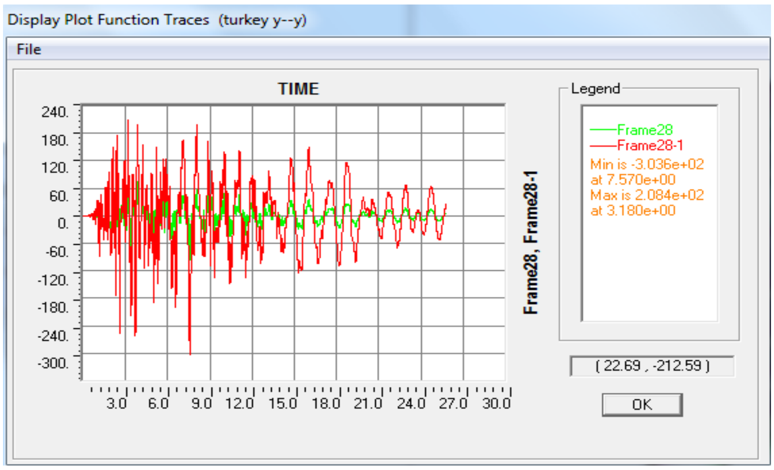


Figure 3.23: The diagram of the variation of the shear force V2 and moment M3 for K10 during Duzce-Turkey x- x .

The absolute values of the maximum shear force V2:

- Turkey X-X: 758 kN/m at 9.36sec
- Turkey Y-Y: 82 kN/m at 7.40 sec

The absolute values of the maximum moment M3:

- Turkey X-X: 3827kNm at 9.36 sec
- Turkey Y-Y: 303 kNm at 7.79 sec.

Moreover the next figures (Figure 3.24 & 3.25) display the maximum displacements of node 452 at the top of the building. The red and the green color illustrate the maximum displacement U1 and U2 respectively, against the time.

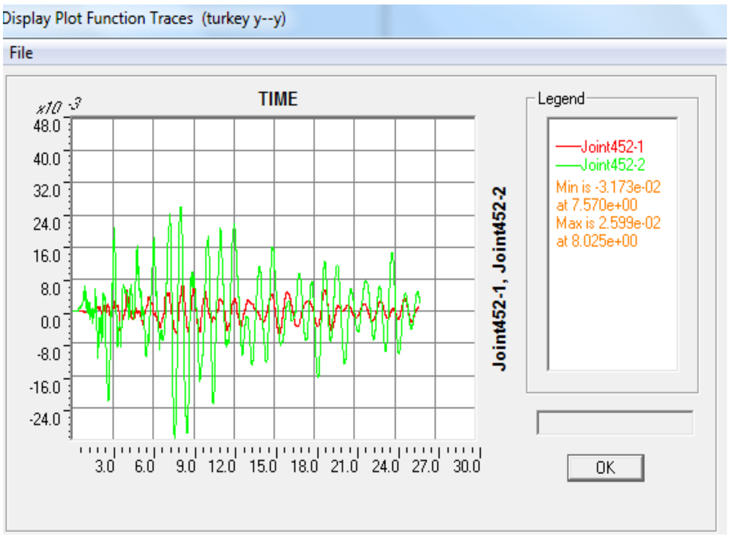


Figure 24: Variation of the maximum displacement U1 and U2 for node 452 during Duzce-Turkey x- x .

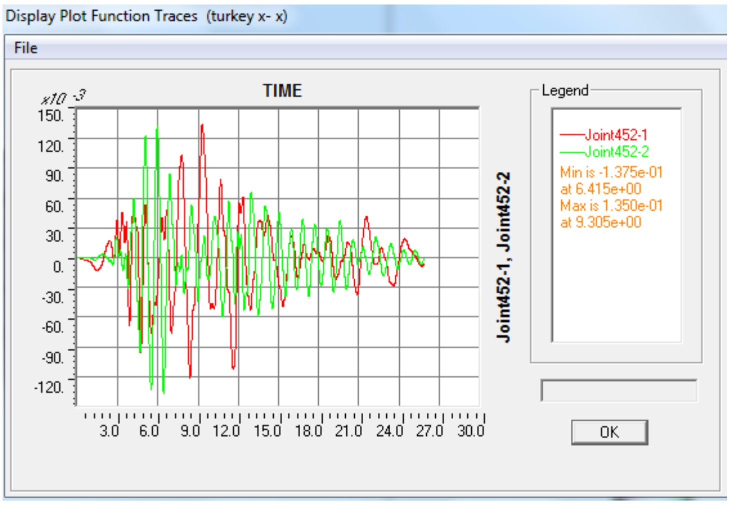


Figure 3.25: Variation of the maximum displacement U1 and U2 for node 452 during Duzce-Turkey y-y .

The absolute values of the maximum displacement UX (U1):

- Turkey-X: 1.34 cm at 9.13 sec
- Turkey Y-Y 0.4 cm at 8.37 sec.

The absolute values of the maximum displacement UY (U2):

- Turkey X-X: 1.26 cm at 6.06 sec
- Turkey Y-Y: 2.92 cm at 8.46 sec

3.5.3 Modeling for time history analysis

- Firstly the two accelerograms in the program will be introduced. The commands are the following**:**

  Define →function → time history → Function type (from file) → add new function (created two accelerograms direction X and Y)

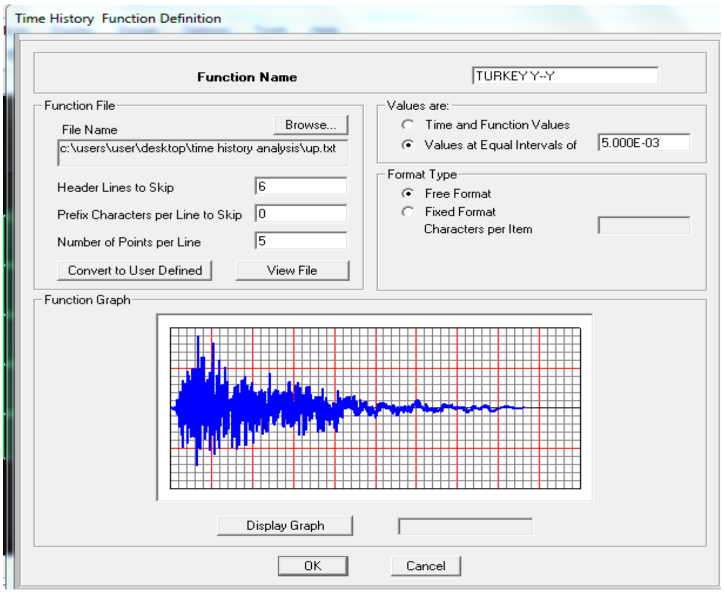


Figure3.26: Import the accelerogram for earthquake (direction Y)

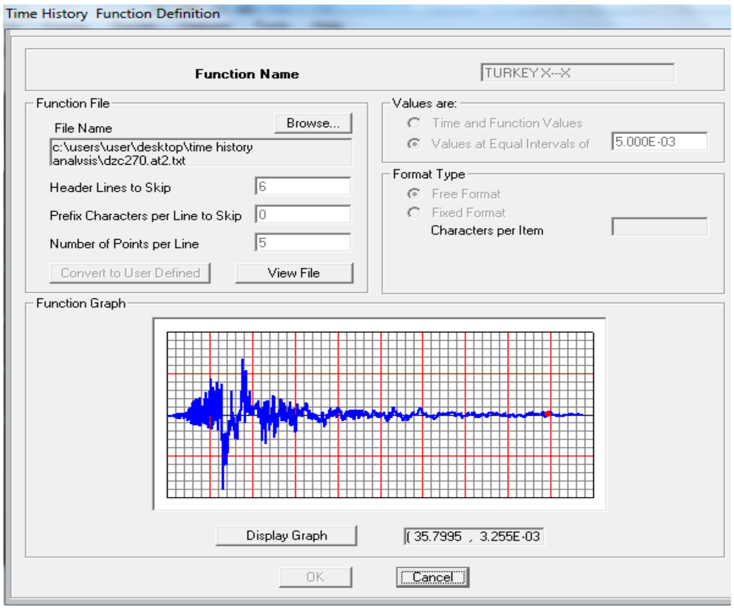


Figure3.27: Import the accelerogram for earthquake (direction X)

- The seismic loading for two directions are determined by following commands :

  Define →load cases → add new load case → load case type (time history)

  - ✓ Scale factor is 9.81 because the accelerogram which has been used had units in g.
  - ✓ In direction X the load name is U1 and the function is Duzce-TURKEY X-X.
  - ✓ In direction Y the load name is U2 and the function is Duzce-TURKEY Y-Y.
  - ✓ Number of time step is equal to 10354 and time step 0.005

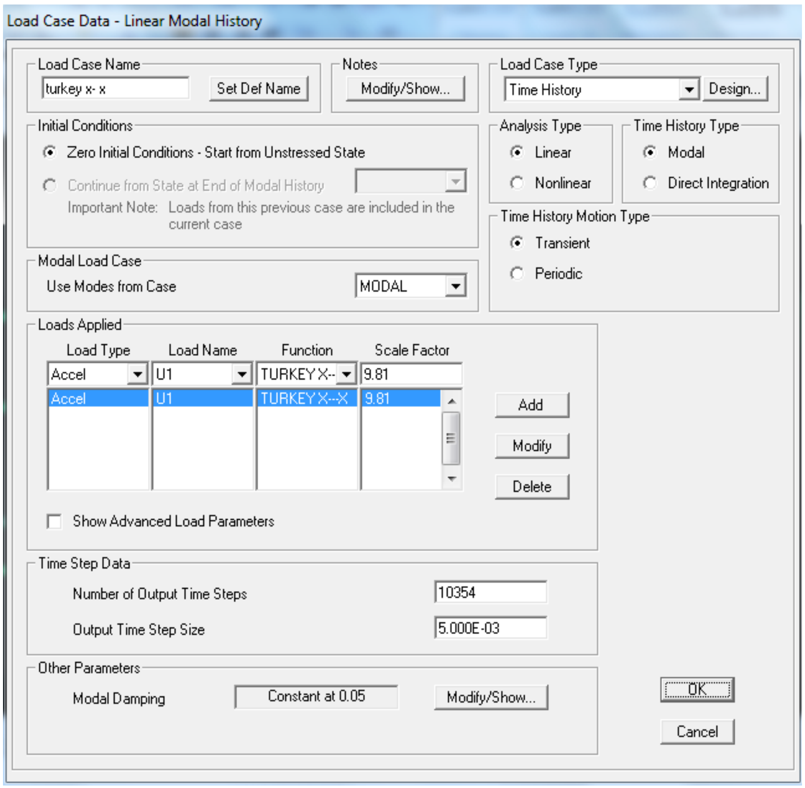


Figure3.28: The seismic loading for X direction is determined

After that the load combination for the two components of the earthquake (X and Y) are created. The commands to achieve this are presented below

Define →load combination → add new combo → load combination type (SRSS)

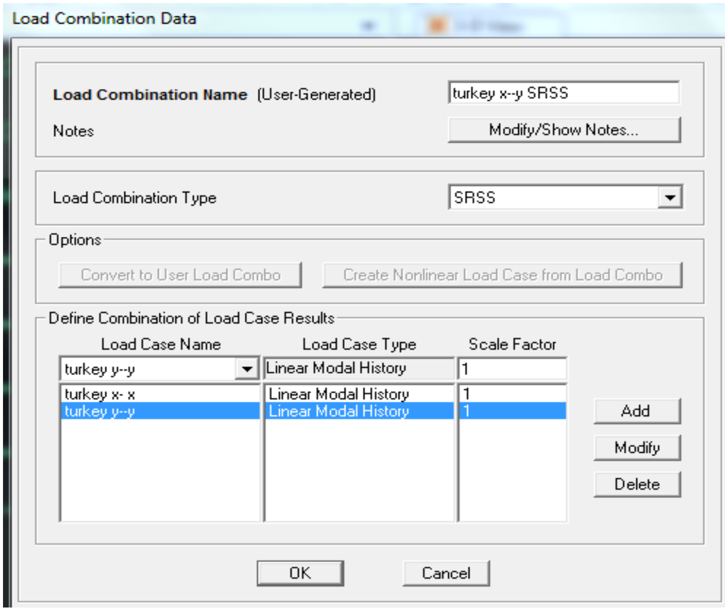


Figure3.29: Load combination for the seismic loading (SRSS)

- The used load combination is: G+ 0.3Q + EX + EY. The two components of the earthquake in X and Y directions are SRSS and the static loads are equal with: G+ 0.3Q. The commands are:

Define →load combination → add new combo → load combination type (Linear add)

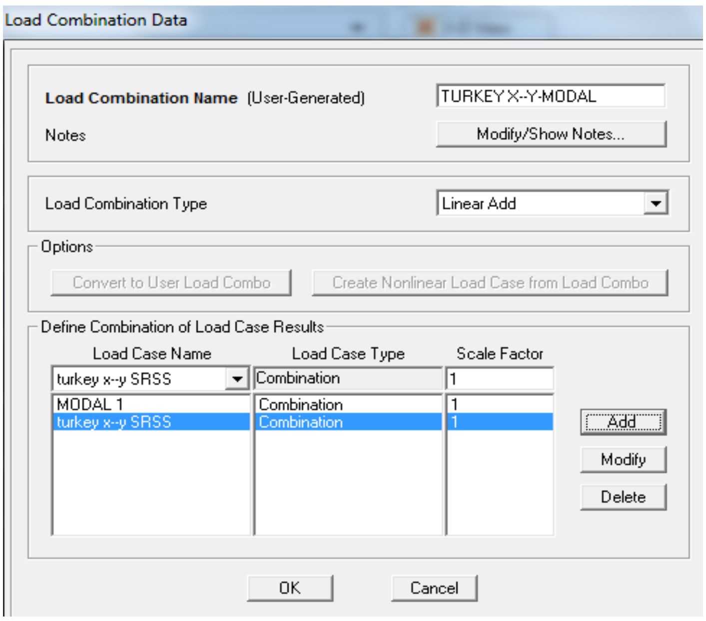


Figure3.30: Load combination for the seismic and static loads

3.6Compare the results of elastic analysis

To compare the results from linear time history analysis with the dynamic spectral method scale factors considered appropriate were used. The response spectrum of seismic excitation is studied as given by the program SeismoSignal refers to the elastic behavior of the construction. Spectrum of EAK is inelastic because the behavior factor q introduces the reduction of seismic acceleration in the real construction. This is a result of the post-elastic behavior of the structure, compare with the acceleration calculated in unlimited elastic system. With the scale factors was approached the inelastic behavior of the structure due to seismic excitation. This is a consequence of the comparison between the values of acceleration for the dominant fundamental period of the structure in the spectrum of design EC-8 with the value of the acceleration response spectrum of the seismic study. Second time history analysis was performed by introducing the corresponding reduction factors of seismic excitation to both directions.

Therefore, the X-direction, has fundamental period Tx = 1,45 sec and the corresponding acceleration for the Turkey spectrum is 2.823 m/s2 .The acceleration of spectrum of EC-8 for a period Tx = 1,45 sec is equal to Sa (Tx) = 0.8126 m/s2.The accelerogram which was introduced in the program (with scaling factor 0.29) is presented below (Figure 3.31).

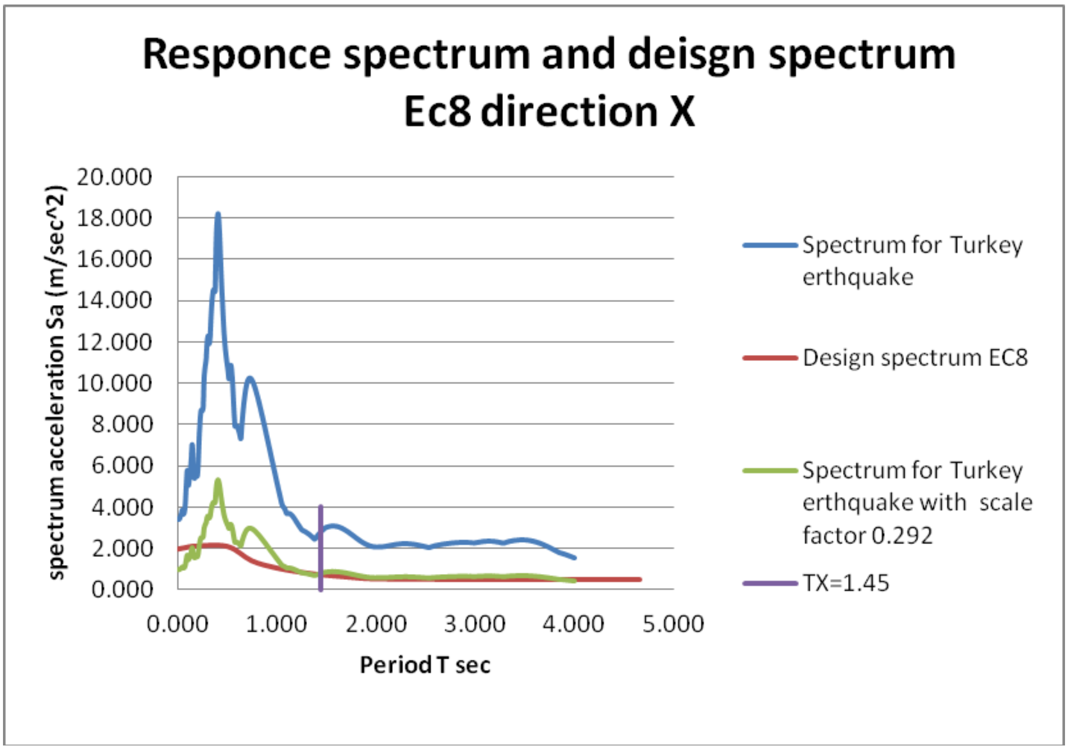


Figure3.31: Response spectrum and design spectrum Ec8 direction x

The Y-direction, has fundamental period Ty = 0.814 sec and the corresponding acceleration for the Turkey spectrum is 1.51 m/s2 .The acceleration of spectrum of EC-8 for a period Ty = 0.814 sec is equal to Sa (Ty) = 1.31 m/s2. The accelerogram which was introduced in the program (with scaling factor 0.872) it can be seen beneath (Figure 3.32).

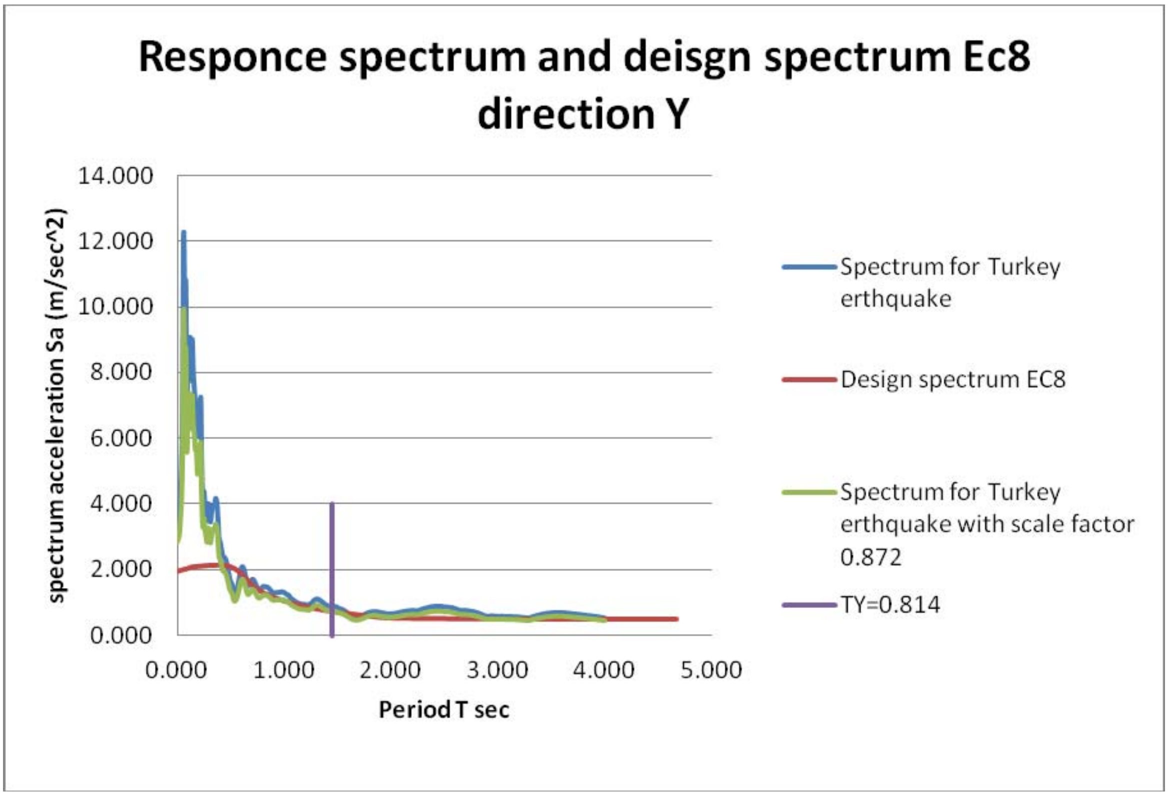


Figure3.32: Response spectrum and design spectrum Ec8 direction y

3.6.1Results for scaled time history analysis

At this part of the study the results of linear time history analysis were performed .The analysis was completed by applying the reducing accelerograms of the earthquake of Duzce in Turkey at 1999 .The above was done to compare the outcome of the results of the two previous methods that were analyzed before. The table 3.8 displays the maximum displacements of the structure derived from the scaled time history analysis.

load combination :	**linear add**	**Max Displacements**				
joint displacement	**U1 (m)**	**U2 (m)**	**U3 (m)**	**R1 (radians)**	**R2(radians)**	**R3(radians)**
ground floor	0.00769	-0.006149	-0.005801	0.003641	0.003854	0.000623
floor 1	0.023979	-0.01921	-0.07428	0.004315	0.004512	0.001903
floor 2	0.040886	-0.032604	-0.007734	0.004124	0.004352	0.003126
floor 3	0.059953	-0.043991	0.008057	0.004015	0.004842	0.004032
floor 4	0.069949	-0.053309	-0.007126	0.002716	0.002796	0.004606
floor 5	0.061836	-0.049808	-0.005808	0.002557	0.001453	0.004606
top floor	0.064322	-0.056085	-0.006091	0.002532	-0.001818	0.004636

Table 3.8: Maximum Displacements according to scaled time history analysis

In the table below it can be observed the maximum values for the axial force, the shear force and the moment for the base column.

		Max Columns' Forces			
floors	**P (KN)**	**V2 (KN)**	**V3(KN)**	**M2(KNm)**	**M3(KNm)**
ground floor	-2336.33	-269.19	684.18	-4112.7	-1155.14
floor 1	-2080.06	-159845.00	639.37	-2316.79	-470.37
floor 2	-1802.94	-116.59	393.64	-1198.63	-418.09
floor 3	-1508.15	-154.980	381.15	-861.15	399
floor 4	-1208.2	118.72	577.33	-1687.76	356.14
floor 5	-933.35	52.142	-856.490	-1687.76	-98.16
top floor	-189.04	118.76	299.13	897.4	194.13

Table 3.9: Maximum forces of the columns according to the scaled time history analysis

The percentage of the comparison between the responses spectral with the scaled time history analysis result is shown in the table beneath (Table 3.10). The results conclude that the time history analysis gives more unfavorable results.

	Time history	Respone Spectru	Time histor	Respone Spectrui	Time history	Respone Spectrum
joint displacemen	**U1 (m)**	**U1 (m)**	**U2 (m)**	**U2 (m)**	**U3 (m)**	**U3 (m)**
ground floor	0.00769	0.004615	-0.006149	-0.004063	-0.005801	-0.004673
percentage	**67%**		**51%**		**24%**	
floor 1	0.023979	0.014388	-0.01921	-0.012945	-0.07428	-0.006123
percentage	**67%**		**48%**		**1113%**	
floor 2	0.040886	0.024408	-0.032604	-0.022457	-0.007734	-0.00666
percentage	**68%**		**45%**		**16%**	
floor 3	0.059953	0.033042	-0.043991	-0.030993	0.008057	-0.00694
percentage	**81%**		**42%**		**186%**	
floor 4	0.069949	0.03912	-0.053309	-0.03823	-0.007126	-0.005985
percentage	**79%**		**39%**		**19%**	
floor 5	0.061836	0.037728	-0.049808	-0.03723	-0.005808	-0.005416
percentage	**64%**		**34%**		**7%**	
top floor	0.064322	0.039139	-0.056085	-0.041973	-0.006091	-0.005661
percentage	**64%**		**34%**		**8%**	

Table 3.10: Maximum Displacements for the response spectrum and scaled time history analysis

From the table above it is observed that the displacements which arise from the analysis with time history are critical. For example the displacement U_1 for the time history analysis at the top floor, is almost double compare with the displacement according to the EC8 response spectrum.

Table 3.11 presents the maximum values for the axial force, the shear force and the moment for the base column according to the two above cases. Furthermore the table above it can be displayed differences between the above two analyses which were mentioned in percentages.

load combination	Time history	Respone Spectru	Time histor	Respone Spectrur	Time history	Respone Spectrum	Time history	Respone Spect	Time history	Respone Spe
floors	P (KN)	P (KN)	V2 (KN)	V2(KN)	V3(KN)	V3(KN)	M2(KNm)	M2(KNm)	M3(KNm)	M3(KNm)
ground floor	-2336.33	-2058.41	-269.19	-210.6	684.18	-487	-4112.7	-3217.67	-1155.14	-912
percentage	**14%**		**22%**		**29%**		**22%**		**21%**	
floor 1	-2080.06	-1788.16	-1598.45	-126.568	639.37	477.32	-2316.79	-1901.33	-470.37	-384.78
percentage	**16%**		**80%**		**25%**		**18%**		**18%**	
floor 2	-1802.94	-1504	-116.59	-108.86	393.64	319.27	-1198.63	-808.54	-418.09	246.72
percentage	**20%**		**7%**		**19%**		**33%**		**41%**	
floor 3	-1508.15	-1218.69	-154.980	-94.63	381.15	202.06	-861.15	-648.76	399	295.368
percentage	**24%**		**39%**		**47%**		**25%**		**42%**	
floor 4	-1208.2	-935.56	118.72	98.45	577.33	366.24	-1687.76	-1300.94	356.14	295.37
percentage	**29%**		**17%**		**37%**		**23%**		**17%**	
floor 5	-933.35	-659.13	52	31.01	-856.490	-559.079	-1687.76	-1300.93	-98.16	-54.9
percentage	**42%**		**41%**		**42%**		**23%**		**44%**	
top floor	-189.04	-180.76	118.76	-48.71	299.13	152.55	897.4	-410.55	194.13	-71.82
percentage	**5%**		**59%**		**49%**		**54%**		**63%**	

Table 3.11 Maximum axial, shear forces and moment for the response spectrum and scaled time history analysis

From the above table (Table 3.11) it can be seen that the results raised from the analysis with time history was scaled are unfavorable. For instance the axial force P1 for the time history analysis at the ground floor is almost double compare with the axial force according to the EC8 response spectrum.

Examining the two tables (Table 3.10 & Table 3.11) it is concluded that the results from the scaled time history analysis are worst in contrast with the results from the response spectrum of EC8. Moreover all the displacements from the time history analysis are almost 40% bigger than them from the spectrum of the EC8. It is also noted that the total axial forces from the time history analysis are approximately 15% larger than the respectively axial forces from the spectrum of EC8. The results from the linear analysis (although they have reduction coefficients) are more crucial.

3.6.2 Modeling for scaled accelerograms

In this part, the accelerograms with scale factor were introduced in the program. The scale factor became 9.81 multiplied by the reduction factor in both directions. The scale factor for the direction X is 0.29 and 0.829 in Y direction as mentioned before.

- The seismic loading for two directions are determined by following commands :

Define →load cases → add new load case → load case type (time history)

- ✓ Direction- x: Scale factor is 9.81*0.29 = 2.845 because the accelerogram which has been used had units in g.
- ✓ Direction- y: Scale factor is 9.81*0.29 = 8.53 because the accelerogram which has been used had units in g.
- ✓ In direction X the load name is U1 and the function is TURKEY X-X.
- ✓ In direction Y the load name is U2 and the function is TURKEY Y-Y.
- ✓ Number of time step is equal to 10354 and time step 0.005

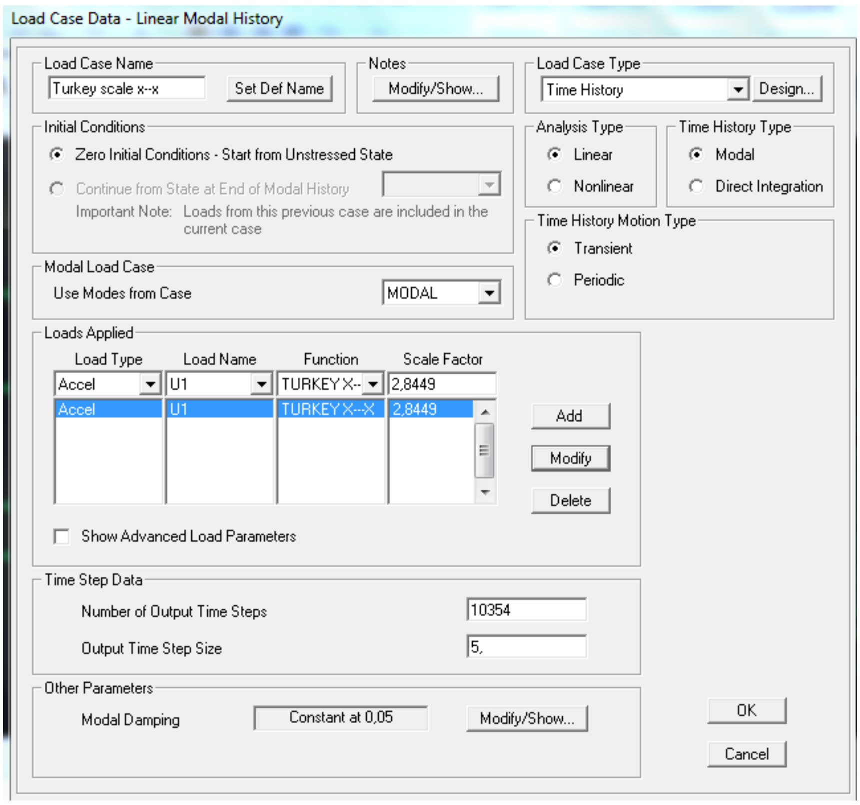


Figure3.33: Introduction of the scaled seismic loading for X direction

- Afterward the load combination was created for two components of the scaled earthquake X and Y. The commands which were used are the following:

Define →load combination → add new combo → load combination type (SRSS)

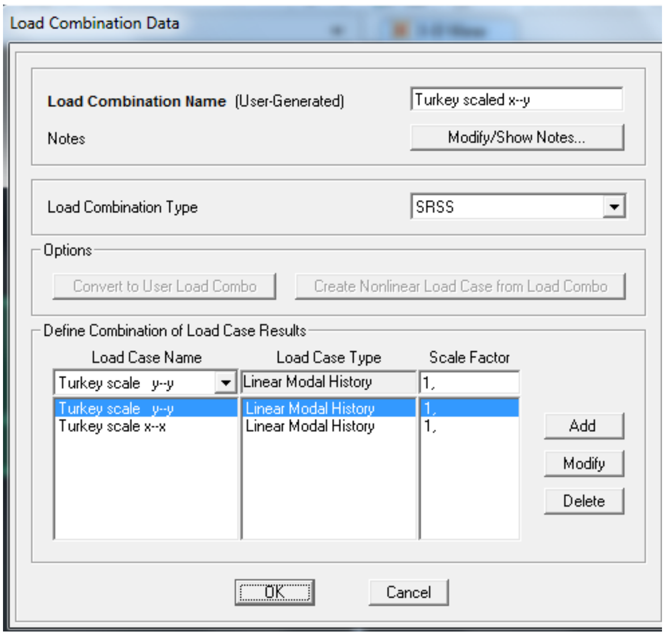


Figure3.34: Load combination for the scaled seismic loading (SRSS)

- The used load combination is G + 0.3Q + EX + EY. The two components of the earthquake in X and in Y directions are SRSS and the statics load is G + 0.3Q. The commands for this procedure are :

Define →load combination → add new combo → load combination type (Linear add)

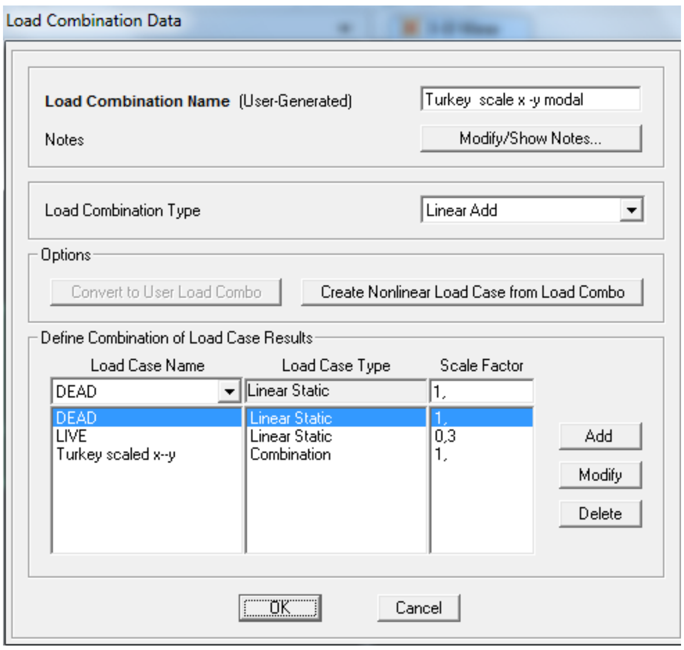


Figure3.35: Load combination for the scaled seismic loading and static loads

4. Performance based design

Introduction

The seismic design of structures with levels of performativity (Performance-Based Design) is based on the principle of establishing an acceptable level damage (level performativity) depending on the probability of seismic vibration design, namely the determination of target seismic capacity. In other words, the method examines the actual way will behave construction at various power levels of seismic vibration design and corresponding expected level of damage. In this way, an optimum combination of safety and economy.

In contrast, the classical design methodology of modern seismic regulations (using the force) only considers the behavior of the structure to begin losses (elastic response) and does not deal with what happens after. The minimum required level of security is ensured through behavior factor used in the study. These factors are determined by behavioral knowledge we have from previous earthquakes and the experimental and analytical research that has been conducted in order to ensure the protection of human life and avoiding collapse. In many cases, however, this design can be precarious, in highly irregular buildings.

4.1The scope of performance based design

In performance-based seismic design of buildings, capacity spectrum technique is an important tool to evaluate the performance point of the structure. Basically, there are two key elements in this method, namely seismic demand and capacity as proposed in ATC-40. The seismic demand is a representation of the earthquake ground motion, and it is presented in terms of forces and displacements imposed on structures by earthquakes.

The seismic capacity represents the inelastic behavior of structure in terms of spectral acceleration and spectral displacement, which is known as capacity curve. The process to determine capacity curve relies on the use of nonlinear static seismic analysis (pushover method). The performance point is defined as the intersection point between demand and capacity where the ductility of structure is matched. This procedure is based on a basic assumption that displacement ductility is a damage criterion.

4.2Description of the Performance level design

Performance-based features of the recent first European Standard for seismic design of buildings (EN1998-1:2004) and of the final draft European Standard for seismic assessment and retrofitting of buildings (prEN1998-3, May 2004) are reviewed, with emphasis on concrete buildings. EN1998-1:2004 includes two performance levels: (a) local collapse endangering lives and (b) limitation of damage in structural and non-structural elements. They are meant to be checked under a rare and an occasional earthquake, respectively, with the definition of the associated seismic hazard levels left to the country. Buildings designed for energy dissipation are protected from global collapse under a very rare (but unspecified) earthquake across-the board application of capacity design to control the inelastic mechanism. The link between the behavior factor q that reduces elastic lateral forces of the (local-) collapse prevention earthquake and member detailing against member collapse is derived. The EN1998-3 provides for 3 performance levels: near collapse, significant damage and limited damage. Verification of ductile members is fully deformation-based. The tools for verification of existing, new or retrofitted members are given as expressions for their limit deformations.

In this dissertation is used the ec8 the prEN1998-3, “Part 3: Assessment and retrofitting of buildings”. Part 3 of Eurocode 8 (CEN 2004b) adopts a fully performance-based approach for existing buildings. Three performance levels (termed “Limit States”) are defined:

*Damage Limitation (*corresponding to “Immediate Occupancy” in the US*)*: The structure is only slightly damaged with insignificant plastic deformations. Repair of structural components is not required, because their resistance capacity and stiffness are not compromised. Cracks may present on non-structural elements, but they can be economically repaired. The residual deformations are unnecessary.

*Significant Damage (*corresponding to “Life safety” in the US): The structure is significantly damaged and it has undergone resistance reduction. The non-structural elements are damaged, yet the partition walls are not failed. The structure consists of permanent significant drifts and generally it is not economic to repair.

*Near Collapse (*similar to “Collapse prevention” in the U.S): The structure is heavily damaged. On the other hand, vertical elements are still able to carry gravity loads. Most non-structural elements are failed, and remained ones will not survive under next seismic actions, even for slight horizontal loads.

Seismic performance of a building is determined by obtaining storey-based structural member damage ratios under a linear or non-linear analysis. Member damage levels are classified as shown in Figure 4.1. The building performances are as in the following:

Immediate Occupancy (IO): For each main direction that seismic loads affect, at any storey at most 10% of beams can be at moderate damage level, however, the rest of the structural elements should be at slight damage level. With the condition of brittle elements to be retrofitted (strengthened), the buildings at this state are assumed to be at *Immediate Occupancy Performance Level*.

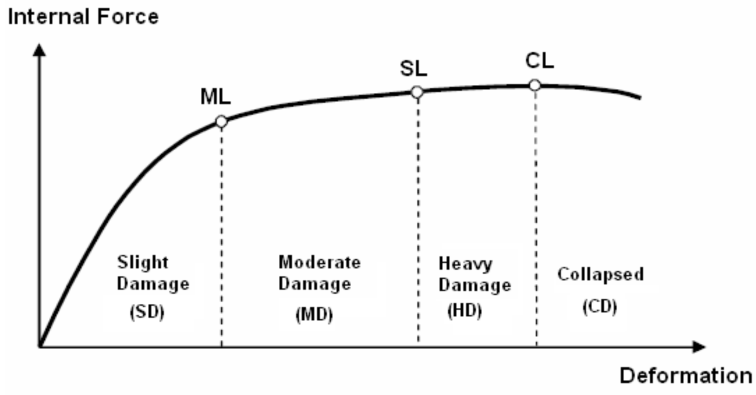


Figure 4.1 :Cross-sectional Member Damage Limits

Life Safety (LS): For each main direction that seismic loads affect, at any storey at most 30% of beams and some of columns can be at heavy damage level, however, shear contributions of overall columns at heavy damage must be lower than 20%. The rest of the structural elements should be at slight or moderate damage levels. With the condition of brittle elements to be retrofitted, buildings at this state are assumed to be at Life Safety Performance Level. For the validity of this performance level, the ratio between the shear force contribution of a column with moderate or higher damage level from both ends and the total shear force of the corresponding storey must be at most 30%. This ratio can be permitted up to 40% at the top storey.

Collapse Prevention (CP): For each main direction that seismic loads affect, at any storey at most 20% of beams can collapse. Rest of the structural elements should be at slight damage, moderate damage, or heavy damage levels. With the condition of brittle elements to be retrofitted, the buildings at this state are assumed to be at Collapse Prevention Performance Level. For the validity of this performance level, the ratio between the shear force contribution of a column with moderate or higher damage level from both ends and the total shear force of the corresponding storey must be at most 30%. Functionality of a building at this performance level has risks for life safety and it should be strengthened. Cost-effective analysis is also recommended for such seismic rehabilitation.

A target performance assessment objective for a given building consists of one or more performance level for given earthquake hazard level. European countries check the return periods due to the various limit states and define it in its National Annex. Recommended return periods to corresponding limit states are given in Table 4.1. Required performance levels to corresponding existing building types are given in Table 4.2.

Limit States	**Return Period**	**Probability of Exceedance**
LS of Damage Limitation	225 years	20% / 50 years
LS of Significant Damage	475 years	10% / 50 years
LS of Near Collapse	2457 years	2% / 50 years

Table4.1: Eurocode 8 Recommended Return Periods

	Probability of Exceedance		
Purpose of Occupancy	**50% / 50 years**	**20% / 50 years**	**2% / 50 years**
Operational After Earthquake	-	IO	LS
Crowded for Long-term	-	IO	LS
Crowded for Short-term	IO	LS	-
Contains Hazardous Material	-	IO	CP
Other	-	LS	-

Table 4.2: Required Seismic Performance Levels

The eurocode itself gives no recommendation, but mentions that the performance objective recommended as suitable to ordinary new buildings is a 225yr earthquake (20% in 50 years), a 475yr event (10% in 50 years), or a 2475yr one (2% in 50 years), for the DL, the SD or the NC “Limit State”, respectively.

4.3 Characteristics of the Performance Based Design

Over the past several years, federal guidelines were published which help to facilitate the implementation of Performance Based Design with respect to existing structures. FEMA 273, guidelines for the Seismic Rehabilitation of Buildings, which has subsequently been updated as FEMA 356, provides specific performance objectives for both the building under consideration and the nonstructural components associated with the building. While written for use with existing structures, the guidelines may also be used as the basis for the design of the seismic force-resisting system for new structures.

Performance Based Seismic Design has the following distinguishing characteristics.

Performance Based Seismic Design allows the owner, architect, and structural engineer to choose both the appropriate level of ground shaking and the chosen level of protection for that ground motion.

Multiple levels of ground shaking can be evaluated, with a different level of performance specified for each level of ground shaking.

Target building performance levels range from Continued Operation, in which the building and nonstructural components are expected to sustain almost no damage in response to the design earthquake, to Collapse Prevention, in which the structure should remain standing, but is extensively damaged.

Specific ductility factors are specified for each component of the seismic force-resisting system. The ductility factor varies depending on the target building performance level, material type, and the relative ductility of the component.

Below (Figure4.2) is a graphical representation of a performance objective matrix that matches chosen earthquake hazard levels (y axis) with selected target building performance levels (x axis). The three diagonal lines represent the performance objectives for different groups of buildings. Group I is representative of a basic commercial structure, while Groups II and III represent structures that require a higher level of protection such as hospitals, fire stations, data centers, key manufacturing facilities, etc.

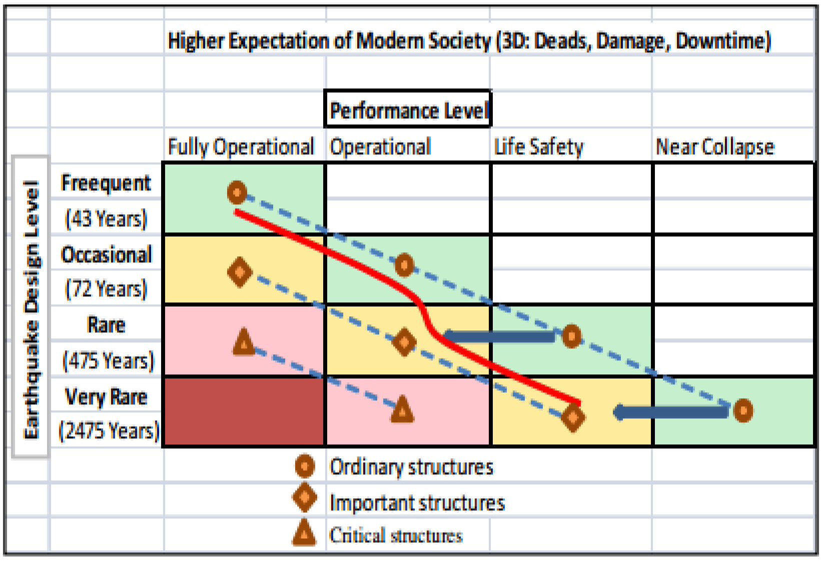


Figure 4.2: representation of a performance objective

4.4 Determination the levels of performativity

4.4.1Pushover analysis (Capacity curve)

Pushover is a static non-linear analysis. The term static implies that a static method is being applied to represent a dynamic phenomenon. The term analysis implies that a pushover is being carried out to an already existing building and evaluates the existing solution and modifies it as needed. Therefore, pushover is a powerful tool for assessment purposes. Non – linear static analysis has been developed extensively over the last years, and the reason is because powerful computers are able to support new computer programs. In the present paper SAP 2000 V15 is used to carry out the pushover analysis to a test building.

A simple example of a pushover analysis is illustrated in Figure 4.3. This procedure requires the execution of a non-linear static analysis of a structure, which allows monitoring progressive yielding of the structure and establishing the capacity curve. The structure is 'pushed' with a lateral loading shape that follows the fundamental mode shape of the pre-yielding building to specific target displacements levels, while vertical earthquake loading is ignored. The resulting plot of Base Shear - Roof Displacement (Figure 4.3) is called the 'capacity curve'. The internal forces and deformations computed at the target displacement levels are estimates of the strength and deformation demands, which need to be compared to available capacities.

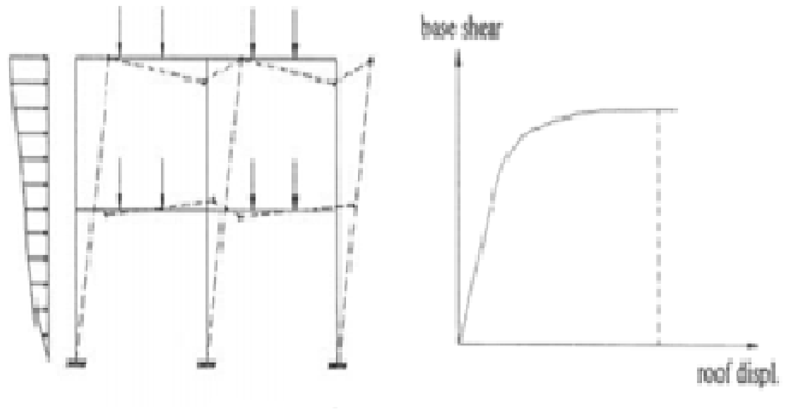


Figure 4.3: Illustration of pushover

Nowadays, some of the newest codes have adopted standards and guidance material regarding the assessment of existing structures and are listed: ATC – 40 (1996), FEMA – 273/274 (1997), FEMA – 356/357 (2000), KANEPE – (2004), EC8 – Part 3, FEMA – 440 (2005), ASCE 41-06 (2007).

4.4.2 Idealized curve F – Δ

The nonlinear procedures require definition of the nonlinear load deformation relation. Such a curve is given in Figure 4.4.

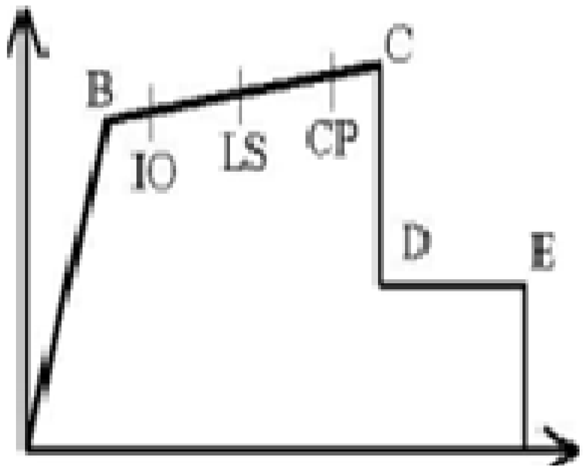


Figure 4.4 Typical load – deformation relation and target performance levels

Point A corresponds to the unloaded condition. Point B corresponds to the nominal steel yield strength. The slope of line BC is usually taken equal to between 0% and 10% of the initial slope (line AB). Point C has resistance equal to the nominal strength. Line CD corresponds to initial failure of the member. It may be associated with phenomena such as fracture of the bending reinforcement, spalling of concrete or shear failure following initial yield. Line DE represents the residual strength of the member. It may be non-zero in some cases, or practically zero in others. Point E corresponds to the deformation limit. However, usually initial failure at C defines the limiting deformation, and in that case point E is a point having deformation equal to that at C and zero resistance.

The five points (A, B, C, D and E) are used to define the hinge rotation behaviour of RC members. Three more points Immediate Occupancy (IO), Life Safety (LS) and (Collapse Prevention) CP, are used to define the acceptance criteria for the hinge.

4.4.3 Plastic hinges

In plastic limit analysis of structural members subjected to bending, it is assumed that an abrupt transition from elastic to ideally plastic behavior occurs at a certain value of moment. This moment is known as plastic moment (M_p). Member behavior between Myp and Mp is considered to be elastic. When Mp is reached, a plastic hinge is formed in the member as it can be observed in Figure 4.5. In contrast to a frictionless hinge permitting free rotation, it is postulated that the plastic hinge allows large rotations to occur a constant plastic moment Mp.

Plastic hinges extend along short lengths of beams. Actual values of these lengths depend on cross – sections and load distributions. On the other hand detailed analyses have shown that it is sufficiently accurate to consider beams rigid – plastic, with plasticity confined to plastic hinges at points. While this assumption is sufficient for limit state analysis, finite element formulations are available to account the spread of plasticity along plastic hinge lengths. By inserting a plastic hinge at a plastic limit load into a statically determinate beam, a kinematic mechanism permitting an unbounded displacement of the system can be formed. It is known as the collapse mechanism. For each degree of static indeterminacy of the beam, an additional plastic hinge must be added to form a collapse mechanism.

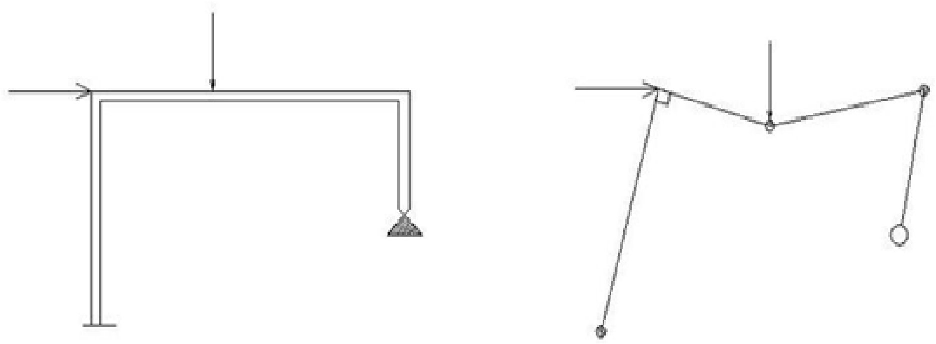


Figure 4.5: Plastic hinges at the ends of the members

Plastic hinges are an extension of the ductile design concept in building seismically resistant structures. Energy is dissipated through the plastic deformation of specific zones at the end of a member without collapsing the rest of the structure. In conventional reinforced concrete columns, this plastic hinge action can result in damage and permanent strain in the column, necessitating replacement of the entire member and possibly the entire structure. However, through the use of specially designed plastic hinge zones, damage due to large seismic displacements can be localized and repaired after an earthquake. In reinforced concrete columns, the detailed plastic hinge consists of a weakened portion of the column near the top and bottom where the longitudinal reinforcement is decreased, allowing yielding in this zone before the rest of the column is damaged. These specially weakened steel bars are termed fuse-bars since they are designed to yield and thus protect the rest of the column during repeated ground motion.

4.4.4 EC 8 Plastic Hinge Rotation Capacities

The deformation capacity of beams, columns and walls, is defined in terms of the chord rotation θ, of the angle between the tangent to the axis at the yielding end and the chord connecting that end with the end of the shear span (the point of contra flexure). The chord rotation is also equal to the element drift ratio, the deflection at the end of the shear span with respect to the tangent to the axis at the yielding end, divided by the shear span.

The state of damage in a structure is defined in EN 1998-3, Eurocode 8, by three limit states, namely Near Collapse (NC), Significant Damage (SD), and Damage Limitation (DL). The structure is heavily damaged, with low residual lateral strength and stiffness, although vertical elements are still capable of sustaining vertical loads. The value of the total chord rotation capacity at ultimate of concrete members under cyclic loading may be calculated from the following expression:

$$\theta_{um} = \frac{1}{\gamma_{el}} 0,016\,(0,3^{v})[\frac{\max(0,01;\omega')}{\max(0,01;\omega)} f_c]^{0,225} (\frac{L_V}{h})^{0,35} 25^{(a\rho_{sx}\frac{f_{yw}}{f_c})} (1,25^{100\rho_d}) \quad [4.1]$$

- The confinement effectiveness factor is:

$$a = (1-\frac{s_h}{2b_0})(1-\frac{s_h}{2h_0})(1-\frac{\sum b_i^2}{6h_0 b_0}) \quad [4.2]$$

- The value of the plastic part of the chord rotation capacity of concrete members under cyclic loading may be calculated from the following expression:

$$\theta_{um}^{pl} = \theta_{um} - \theta_y = \frac{1}{\gamma_{el}} 0,0145\,(0,25^{v})[\frac{\max(0,01;\omega')}{\max(0,01;\omega)}]^{0,3} f_c^{\,0,2} (\frac{L_V}{h})^{0,35} 25^{(a\rho_{sx}\frac{f_{yw}}{f_c})} (1,275^{100\rho_d}) \quad [4.3]$$

In members without detailing for earthquake resistance the values given by equations 4.1 & 4. 3 are multiplied by 0,825.

In members with smooth (plain) longitudinal bars θum given by equation 4.1 is multiplied by 0,575, while Θum given by equation 4.3 is multiplied by 0,375 (where these factors include the reduction factor of 0,825 given above).

For the evaluation of the ultimate chord rotation capacity EC 8 proposes also an alternative expression to equation 4.1:

$$\Theta_{um} = \frac{1}{\gamma_{el}}(\Theta_y + (\varphi_u - \varphi_y)L_{pl}(1 - \frac{0,5L_{pl}}{L_v})) \quad [4.4]$$

The value of the length Lpl of the plastic hinge depends on how the enhancement of strength and deformation capacity of concrete due to confinement is taken into account in the calculation of φu. For the evaluation of the length Lpl expression to equation:

$$L_{pl} = 0{,}1L_V + 0{,}17h + 0{,}24\frac{d_{bL}f_y(\text{MPa})}{\sqrt{f_c(\text{MPa})}} \quad [4.5]$$

There are two procedures given for this in EC8.

- <u>Limit State of Significant Damage (SD)</u>

The structure is significantly damaged, with some residual lateral strength and stiffness, and vertical elements are capable of sustaining vertical loads. The chord rotation capacity corresponding to significant damage θ(SD) may be assumed to be 75% of the ultimate chord rotation θu given by Equation 4.1.

➢ Limit State of Damage Limitation (DL)

The structure is only lightly damaged, with structural elements prevented from significant yielding and retaining their strength and stiffness properties. The capacity for this limit state used in the verifications is the yielding bending moment under the design value of the axial load. In case the verification is carried out in terms of deformations the corresponding capacity is given by the chord rotation at yielding θy, evaluated for beams and columns using the following equation:

$$\theta_y = \varphi_y \frac{L_V + a_V z}{3} + 0{,}00135\left(1 + 1{,}5\frac{h}{L_V}\right) + \frac{\varepsilon_y}{d - d'}\frac{d_b f_y}{6\sqrt{f_c}} \qquad [4.6]$$

An alternative expression is:

$$\theta_y = \varphi_y \frac{L_V + a_V z}{3} + 0{,}0013\left(1 + 1{,}5\frac{h}{L_V}\right) + 0{,}13\varphi_y \frac{d_b f_y}{\sqrt{f_c}} \qquad [4.7]$$

The first term in the above expressions accounts for flexure, the second term for shear deformation and the third for anchorage slip of bars.

4.4.5 Description of hinges in Sap2000

In SAP2000, a frame element is modeled as a line element having linearly elastic properties and nonlinear force-displacement characteristics of individual frame elements are modeled as hinges represented by a series of straight line segments. A generalized force-displacement characteristic of a non-degrading frame element (or hinge properties) in SAP2000. Fig. 4.4 Force-Deformation for Pushover Hinge shows that point A corresponds to unloaded condition and point B represents yielding of the element. The ordinate at C corresponds to nominal strength and abscissa at C corresponds to the deformation at which significant strength degradation begins. The drop from C to D represents the initial failure of the element and resistance to lateral loads beyond point C is usually unreliable. The residual resistance from D to E allows the frame elements to sustain gravity loads. Beyond point E, the maximum deformation capacity, gravity load can no longer be sustained.

Hinges can be assigned at any number of locations (potential yielding points) along the span of the frame element as well as the element ends. Uncoupled moment (M_2 and M_3), torsion (T), axial force (P) and shear (V_2 and V_3) force – displacement relations can be defined. As the column axial load changes under lateral loading, there is also a coupled P – M_2 – M_3 (PMM) hinge which yields. This hinge is based on the interaction of axial force and bending moments at the hinge location. Moreover, more than one type of hinge can be assigned at the same location of a frame element. There are three types of hinge properties in SAP2000. These are:

- the default hinge properties,
- the user-defined hinge properties and
- the generated hinge properties.

Only the default hinge properties and the user-defined hinge properties can be assigned to frame elements. When these two hinge properties (default and user-defined) are assigned to a frame element, the program automatically creates a new generated hinge property for every hinge.

Default hinge properties could not be modified and they are dependent from the section. When the default hinge 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. The built – in default hinge properties for steel and concrete members are based on ATC – 40 and FEMA – 273 criteria.

The user – defined hinge properties can be based on the default properties or they can be fully user – defined. When user – defined properties are not based on default properties, then the properties can be viewed and modified. The generated hinge properties are used in the analysis. They could be viewed, but they could not be modified.

5. Non-Linear Analysis

5.1 Introduction

Structures are allowed to exhibit significant inelastic deformation under a strong earthquake. The dynamic characteristics of a structure change with time, so to investigate the performance of it requires inelastic analytical procedures based on these features.

Inelastic analytical procedures help to understand the actual behavior of structures by identifying failure modes and the potential for progressive collapse. Inelastic analysis procedures basically include inelastic time history analysis and inelastic static analysis (which is also known as pushover analysis).

The inelastic time history analysis is the most accurate method to predict the force and the deformation of the components in a structure. However, the use of it is limited because the dynamic response is very sensitive to modeling and ground motion characteristics. It needs proper modeling of cyclic load deformation characteristics taking account the deterioration properties of all important components. Furthermore, it requires the availability of a set of representative ground motion records that accounts for uncertainties and differences in severity, frequency and duration. Moreover, the computation time (time required for the preparation of input and interpreting voluminous output) make the use of inelastic time history analysis impractical for seismic performance evaluation.

Inelastic static analysis commonly referred to as “push over” analysis, is used to determine the reliable displacement capacities of a structure or frame as it reaches its limit of structural stability. It is performed by using the expected material properties of modeled members. Inelastic static analysis is an incremental linear analysis, which captures the overall nonlinear behavior of the elements, including soil effects, by pushing them laterally to initiate plastic action. Each increment pushes the frame laterally, through all possible stages, until the potential mechanism collapse. The internal actions, as components, respond inelastically due to the fact that the analytical model accounts for the redistribution. Inelastic static analysis is expected to provide a more realistic measure of behavior than can be obtained from elastic analysis. The theoretical background, the reliability and the accuracy of inelastic static analysis is going to discussed in detail in the following sections

Scope: Although an elastic analysis gives a good indication of the elastic capacity of structures and indicates where first yielding will occur, it cannot predict the failure mechanisms and take in account the redistribution of forces during progressive yielding. On the other hand inelastic analysis helps to demonstrate the way which buildings act in reality by identifying modes of failure and the potential for progressive collapse. The main use of inelastic procedures for design and evaluation is to understand how the structures will behave when subjected to major earthquakes. In such cases it is assumed that the elastic capacity of a structure will be exceeded. This assumption resolves some of the uncertainties associated with code and elastic procedures.

5.1.1Description of pushover analysis

Pushover analysis is an approximate analysis method in where the structure is subjected to monotonically increasing lateral forces with an invariant height-wise distribution until a target displacement is reached. 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 action uses a series of sequential elastic analyses, which is superimposed to approximate a force-displacement capacity diagram of the overall structure. The mathematical model of the structure is modified to calculate the reduced resistance of yielding components.

A lateral force distribution is applied until additional components yield. These lateral forces apply in proportion to the product of story masses and first mode shape of the elastic model .The capacity curve is generally constructed to represent the first mode response of the structure based on the assumption that the fundamental mode of vibration is the predominant response of the structure. This is generally valid for buildings with fundamental periods of vibration up to about one second. For more flexible buildings with a fundamental period greater than one second, the analyst should consider addressing higher mode effects in analysis

The process continues until a control displacement, at the top of building reaches a certain level of deformation or until the structure becomes unstable. The roof displacement is plotted with base shear to get the global capacity curve. Figure 5.1 presents a typical diagram for a base shear with the respectively roof displacement.

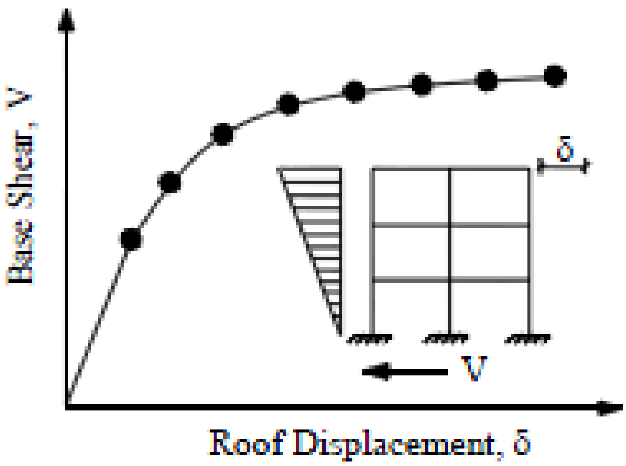


Figure 5.1: Push over curve of structure

Pushover analysis can be performed as force-controlled or as displacement-controlled. In force-controlled pushover full load combination is applied as specified. It is mainly used when the load is known (such as gravity loading).Also, in this procedure some numerical problems that affect the accuracy of results occur as the target displacement may be associated with every small positive or even a negative lateral stiffness because of the development of mechanisms and P-delta effects. In displacement-controlled procedure, specified drifts are sought (as in seismic loading) where the magnitude of applied load is not known in advance. The magnitude of load combination is increased or decreased as necessary until the control displacement reaches a specified value.

Generally, roof displacement at the center of mass of structure is chosen as the control displacement. The calculated internal forces and deformations at the target displacement are used to estimate the demanded inelastic strength and deformation that have to be compared with the available capacities for a performance check. Sometimes is preferred to continue the construction of the capacity curve beyond the above suggested stopping point to observe the structural behavior assuming that all inadequate elements are retrofitted.

5.1.2 Use of Pushover Results

Pushover analysis has been the preferred method for seismic performance evaluation of structures by the major rehabilitation guidelines and codes because it is conceptually and computationally simple. It allows tracing the sequence of yielding and the failure on member. The structural level, as well as the progress of overall capacity, curves of the structure.

The prospect from pushover analysis is to estimate critical response parameters imposed on structural system and its components as close as possible to those predicted by nonlinear dynamic analysis. It provides information on many response characteristics that cannot be obtained from an elastic static or elastic dynamic analysis. This information is:

• Estimation of interstory drifts and its distribution along the height.

• Determination of force demands on brittle members, such as axial force demands on columns, moment demands on beam-column connections.

• Determination of deformation demands for ductile members.

• Identification of location of weak points in the structure (or potential failure modes).

• Consequences of strength deterioration of individual members on the behavior of structural system.

• Identification of strength discontinuities in plan or elevation that will lead to changes in dynamic characteristics in the inelastic range.

• Verification of the completeness and adequacy of load path.

Pushover analysis also exposes design weaknesses that may remain hidden in an elastic analysis. These are story mechanisms, excessive deformation demands, strength irregularities and overloads on potentially brittle members.

5.1.3 Limitations of Pushover Analysis

Although pushover analysis has advantages in contrast with the elastic analysis, underlying assumptions, the accuracy of pushover predictions and limitations of current pushover procedures must be identified. The estimate of target displacement, selection of lateral load patterns and identification of failure mechanisms due to higher modes of vibration are important issues that affect the accuracy of pushover results. Target displacement is the global displacement, which is expected in a design earthquake. The roof displacement at mass center of the structure is used as target displacement. The accurate estimation of target displacement, associated with specific performance objective, have an influence on the accuracy of seismic demand predictions.

In this analysis, the target displacement for a multi degree of freedom (MDOF) system is usually estimated as the displacement demand for the corresponding equivalent single degree of freedom (SDOF) system. The basic properties of an equivalent SDOF system are obtained by using a shape vector which represents the deflected shape of the MDOF system. Most of the researchers suggest the use of normalized displacement profile at the target displacement level as a shape vector but iteration is needed since this displacement is not known a priori. Thus, a fixed shape vector, elastic first mode, is used for simplicity without regards to higher modes by most of the approaches. Moreover, hysteretic characteristics of MDOF should be incorporated into the equivalent SDOF model. This is happening when the displacement demand is affected from stiffness degradation or pinching, strength deterioration and P-ΔEffects. Foundation uplift, tensional effects and semi-rigid diaphragms are also expected to affect the target displacement.

Lateral loads represent the likely distribution of inertia forces imposed on structure during an earthquake. The distribution of inertia forces vary with the severity of earthquake and with time during earthquake since:

$$F_i = V\,\frac{m_i\varphi_i}{\sum m_i\varphi_i}$$

$V = \sum F_i$ is the base shear force. The coefficient φ_i is the distribution of displacement at the floors and is usually taken equal to the corresponding values of the first mode.

However, in pushover analysis, generally an invariant lateral load pattern is used when the distribution of inertia forces is assumed to be constant during earthquake. In this case the deformed configuration of structure under the action of invariant lateral load pattern is expected to be similar to the design earthquake. The selection of the lateral load pattern is more critical than the accurate estimation of target displacement because of the response of the structure (the capacity curve is very sensitive to the choice of lateral load distribution).

The lateral load patterns used in pushover analysis are proportional to product of story mass and displacement associated with a shape vector at the story under consideration. Commonly used lateral force patterns are uniform, elastic first mode, "code" distributions and a single concentrated horizontal force at the top of structure. Multi-modal load pattern derived from Square Root of Sum of Squares (SRSS) story shears is also used to consider at least elastic higher mode effects for long period structures. These loading patterns usually favor certain deformation modes that are triggered by the load pattern. They also miss others that are initiated and propagated by the ground motion and inelastic dynamic response characteristics of the structure.

Moreover, invariant lateral load patterns could not predict potential failure modes due to middle or upper story mechanisms caused by higher mode effects. Invariant load patterns can provide adequate predictions if the structural response is not severely by higher modes and the structure has only a single load yielding mechanism that can be captured by it.

FEMA-273 recommends utilizing at least two fixed load patterns that form upper and lower bounds for inertia force distributions to predict likely variations on overall structural behavior and local demands. The first pattern should be uniform load distribution and the other should be "code" profile or multi-modal load pattern. The 'Code' lateral load pattern is allowed if more than 75% of the total mass participates in the fundamental load.

The invariant load patterns cannot relate with the redistribution of inertia forces, because the progressive yielding and resulting changes in dynamic properties of the structure. Also, fixed load patterns have limited capability to predict higher mode effects in post-elastic range. These limitations have led many researchers to propose adaptive load patterns which consider the changes in inertia forces with the level of inelasticity. The underlying approach of this technique is to redistribute the lateral load shape with the extent of inelastic deformations. Even though some improved predictions have been obtained from adaptive load patterns, they make pushover analysis computationally demanding and conceptually complicated. The scale of improvement has been a subject of discussion that simple invariant load patterns are widely preferred at the expense of accuracy.

Whether lateral loading is invariant or adaptive, it is applied to the structure statically that a static loading cannot represent inelastic dynamic response with a large degree of accuracy. The above discussion on target displacement and lateral load pattern reveals that pushover analysis supposes that the response of a building can be related to that of an equivalent SDOF system. In other words, the response is controlled by fundamental mode which remains constant throughout the response history without considering progressive yielding.

5.2 Determination of performance point with the Capacity spectrum method

5.2.1 General

The generalized nonlinear static analytical procedure (Pushover) is a key element in the methodology introduced by ATC-40 for the seismic evaluation and retrofit design of existing buildings represented a fundamental change for the structural engineering profession. The methodology is performance-based where the design criteria are expressed as Performance objectives. These objectives define desired levels of seismic performance when the building is subjected to specified levels of seismic ground motion. This analysis has three primary elements (Figure 5.2): capacity curve of a structure by the use of a static pushover analysis, a method to determine displacement demand by the use of reduced demand spectra, and the resulting identification of the performance point with the subsequent check for acceptable performance.

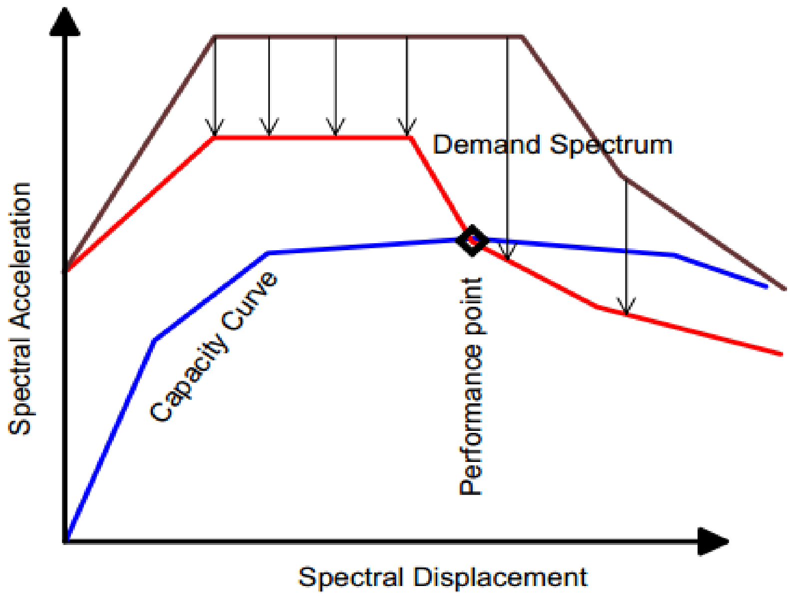


Figure 5.2: The procedure for the identification of the performance point of the structure

The capacity spectrum method was developed to evaluate a structure by comparing the seismic capacity with the seismic demand in the context of non linear static analysis .A capacity curve is created by plotting the total lateral seismic shear (applied to the structure at various increments of loading) with the lateral displacement of given portion. The demand curve is generally represents the modify form of an earthquake response spectrum.

The capacity spectrum method by ATC-40 is based on idealistic hysteric models for the structure and spectra are modified on various equivalent- damping ratios. The use of this method, the capacity curve (which relates the base shear to the roof displacements) and the demand response spectrum are converted into acceleration displacement response spectra format. Both curves are plotted as spectra acceleration among spectral displacement. The performance point is determined as the intersection of the capacity spectrum and the reduced seismic demand curve.

5.2.2 Description of the method

The location of the Performance Point must satisfy two relationships:

1) The point must lie on the capacity spectrum curve in order to represent the structure at a given displacement.

2) The point must lie on a spectral demand curve, reduced from the elastic, 5 percent-damped design spectrum that represents the nonlinear demand at the same structural displacement.

For this methodology, spectral reduction factors are given in terms of effective damping. An approximate effective damping is calculated based on the shape of the capacity curve, the estimated displacement demand, and the resulting hysteresis loop. Probable imperfections in real building hysteresis loops, including degradation and duration effects, are accounted for by reductions in theoretically calculated equivalent viscous damping values. In the general case, determination of the performance point requires a trial and error search' for satisfaction of the two criterion specified above.

5.2.3 Conversion of the nonlinear system to an equivalent linear

The estimation of maximum inelastic displacement demand of MDOF structure from the maximum displacement demand of corresponding equivalent SDOF system forms the underlying principle of most of the proposed approximate procedures. The valuation methods and control of seismic response of structures that have been developed and are detailed below convert inelastic dynamic problem involving complex models inelastic static in order to finally extract the available capacity. An important role for making a non-linear system to an equivalent linear is the viscous damping. The viscous damping is a quantity depends on the energy absorption of the equivalent inelastic and elastic system during the cyclic loading.

A crucial parameter in this conversion is the choice of the distribution of horizontal loads at the floor. The load distribution is proportional to the distribution of the forces of inertia and hence directly related to the masses of each floor on the below relationship:

$$F_i = V\,\frac{m_i\varphi_i}{\sum m_j\varphi_j}$$

The $V = \sum F_i$ is the base shear force. The coefficientsφ_i, illustrate the distribution of displacement at the floors and are usually taken equal to the corresponding values of the first mode. However, note that instead of the first mode could be used any other distribution of deformations, representative of expected deformation of the structure. Usually, the values of φ_i, are normalized so that the value of the peak is equal $\varphi_{top} = 1$.

If the distribution of loads is made in cooperation the equation and$\varphi_{top} = 1$ correlation between the multistage system and the equivalent SDOF sizes (forces, transportation, energy, etc.) is done with the relationship:

$$Q = \Gamma \times Q^*$$

- Q^*Size equivalent single-degree of freedom system
- Q Size equivalent multi-degree of freedom system
- Γ coefficient of participation is given by :

$$\Gamma = V\,\frac{\sum m_i\varphi_i}{\sum m_i\varphi t^2}$$

Since both forces and displacements follow the same rule of transformation the stiffness of equivalent SDOF system is equal to that of multi degree of freedom system .The period of the equivalent SDOF system is not equal with the first period of the multi degree of freedom system even if the coefficient φ_i are equal with the corresponding values of the eigenvector.

To use the capacity spectrum method it is necessary to convert the capacity curve, which is in terms of base shear and roof displacement, to what is called a capacity spectrum. A capacity spectrum is a presentation of the capacity curve in Acceleration-Displacement Response Spectra (ADRS) format. The figure which follows (Figure 5.3) presents this transformation:

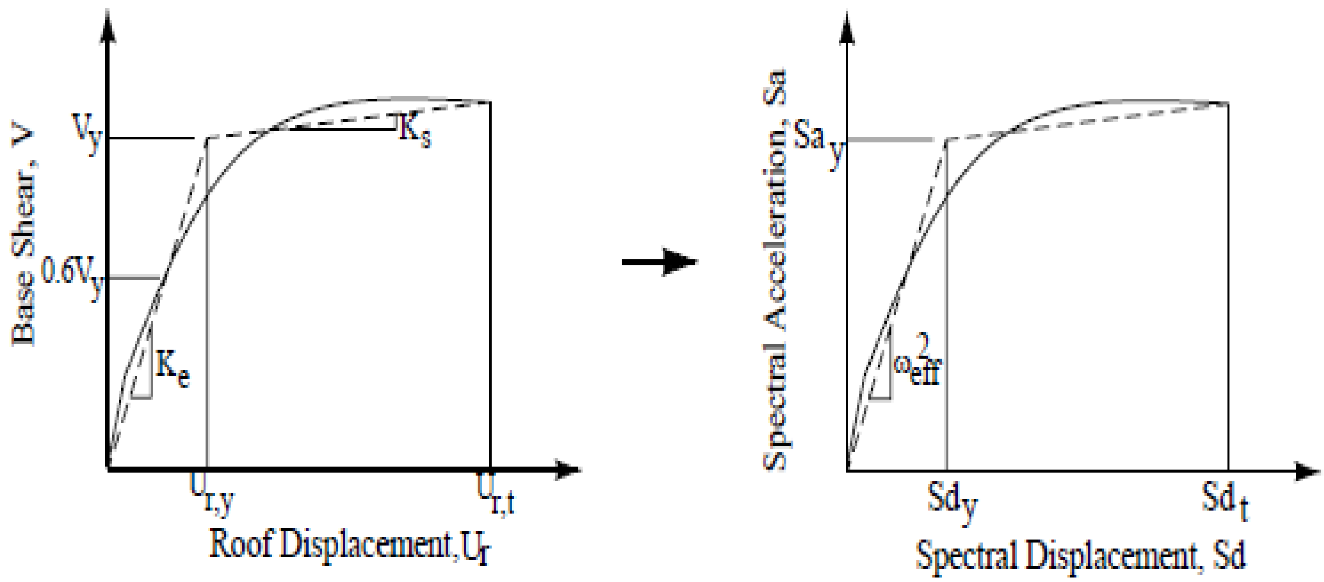


Figure 5.3: Conversion of the capacity curve into acceleration-displacement response spectra format

The required equations to make the transformation are:

$$S_a = \frac{V}{a\, m_{total}}$$

$$S_d = \frac{\Delta}{\Gamma \varphi_{top}} \qquad S_d = \frac{\Delta}{\Gamma} \qquad \text{if } \varphi\text{top} \neq 1$$

- m: total mass of building
- V: base shear (kN)
- Δ : roof displacement (m)

- α : modal mass coefficient for the fundamental mode .The percentage of the total mass involved in the dynamic response of the structure for the expected form deformation, given by the relation:

$$\alpha = \frac{[\sum m_i \varphi_i]^2}{m_{total} \sum m_i \varphi i^2} = \frac{\Gamma \sum m_i \varphi_i}{m_{total}} = \Gamma \frac{m^*}{m_{total}}$$

Typical values of a coefficient for various behavioral of buildings are shown below:

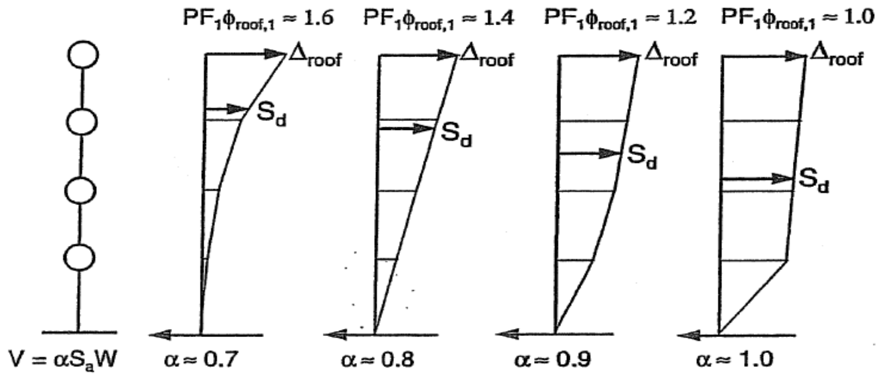


Figure 5. 4: Values of a coefficient for various behavioral of buildings

5.2.4 Capacity spectrum method

The capacity spectrum method by ATC-40 is based on idealistic hysteric models. The construction and the spectra are modified on various equivalents - damping ratios. The use of the ATC-40 capacity spectrum method, the capacity curve (which relates base shear to roof displacements) and the demand response spectrum are converted into acceleration displacement response spectra format. Both curves are plotted as spectra acceleration with spectral displacement. The performance point is determined as the intersection of the capacity spectrum and the reduced seismic demand curve.

ATC-40 displays the recent three versions of the Capacity Spectrum Method for estimation the earthquake induced displacement demand of inelastic systems. All three procedures are based on the same underlying principles that have assumptions as they avoid the dynamic analysis of inelastic system. Instead, the displacement demand of inelastic system is estimated by dynamic analysis of a series of equivalent linear systems with successively updated values of Teq and ζeq.

Processes A and B are analytical and suitable to computer implementation while C is graphical and more suitable for hand analysis. In this case, the procedure which is equivalent to Procedure A in ATC-40, systems was utilized. It contains the following steps:

- Step 1: Develop a capacity curve (base shear versus roof displacement) of the overall building with the usage of pushover analysis.
- Step 2: Construct a bilinear representation of capacity curve.

The approach used in Displacement Coefficient Method was utilized to construct the bilinear representation of capacity curve. In this approach, a line representing the average post-elastic stiffness, Ks, of capacity curve is first drawn by judgment. Then, a secant line representing effective elastic stiffness, Ke, is done. This line intersects the capacity curve at 60% of the yield base shear. The yield base shear, Vy, is defined at the joint of Ke and Ks lines. The process is iterative because the value of yield base shear is not known at the beginning. An illustrative capacity curve and its bilinear representation can be seen underneath (Figure 5.5).

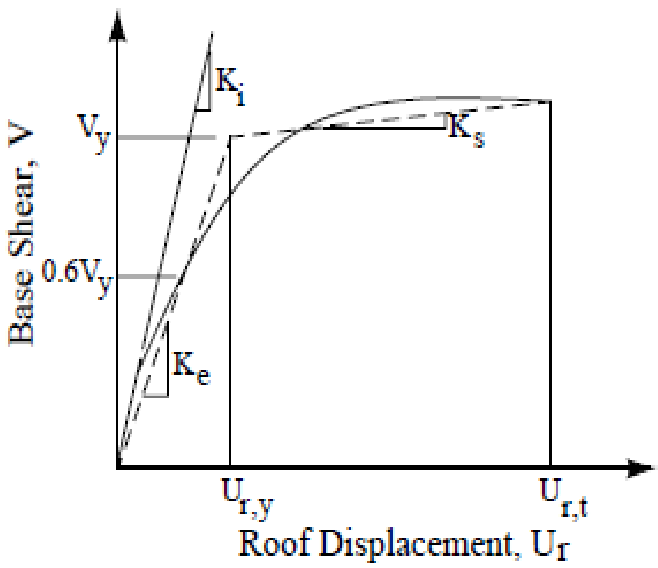


Figure 5.5: Bilinear representation of the capacity curve

5.2.4.1 Convert 5% elastic response (demand) spectrum from standard SA vs. T format to Sa vs. Sd (ADRS) format

Most engineers are familiar with the traditional Sa versus T representation of response spectra. However, they are less familiar with the Sa versus Sd (ADRSr representation). Figure below (Figure 5.6) shows the same spectrum in each format. In the ADRS format lines, radiating from the origin, have constant period. For any point on the ADRS spectrum, the period T, can be found using the relationship: $T = 2\pi\sqrt{\frac{S_d}{S_a}}$.

Similarly, for any point on the traditional spectrum, the spectral displacement, Sd, can be computed using the relationship: $S_d = \frac{1}{4\pi^2} S_a T^2$. These two relationships are the same formula arranged in different ways.

Where:

Sa spectral acceleration (m/s^2)

Sd spectral displacement (m)

T period (s)

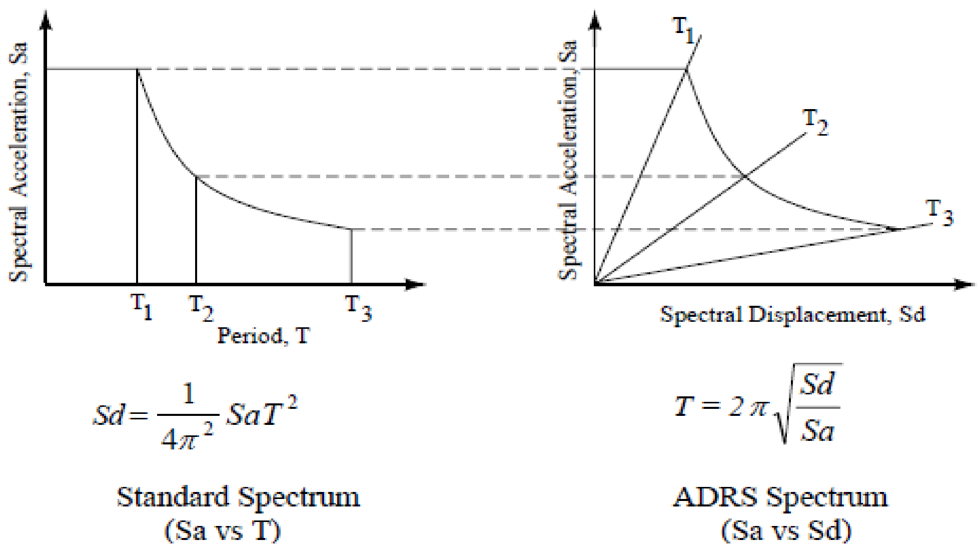


Figure 5.6: Representation SA versus T and SA versus Sd of response spectra

5.2.4.2 Bilinear Representation of Capacity Spectrum

A bilinear representation of the capacity spectrum is needed to estimate the effective damping and the appropriate reduction of spectral demand. The creation of the bilinear representation requires definition of the point a_{p_i}, d_{p_i}. This point is the trial performance point which develops a reduced demand response spectrum. If the reduced response spectrum is found to intersect the capacity spectrum at the estimated a_{p_i}, d_{p_i} point, then that point is the performance point. The first estimation of point a_{p_i}, d_{p_i} is designated at a_{p_1}, d_{p_1}, the second at a_{p_2}, d_{p_2}, and the others have the same pattern. Guidance on a first estimate of point a_{p_1}, d_{p_1} is given in the step-by-step process for each of the three processes. Sometimes, the equal displacement approximation can be used as an approximation of a_{p_1}, d_{p_1}. Refer to the next figure (Figure 5.7) for a bilinear representation of a capacity spectrum example. To construct the bilinear representation one line up is drawn from the origin at the initial stiffness of the building using element stiffness. Then a second line is made back from the trial performance point, a_{p_i}, d_{p_i}. After that the second line is sloped and it intersects the first line. At point ay, dy, the area designated A1 in the figure is approximately equal to the area designated A2. The importance of the setting area A1 is the same to the area A2 is to have equivalent area under the capacity spectrum and its bilinear representation, that is, to have equal energy associated with each curve.

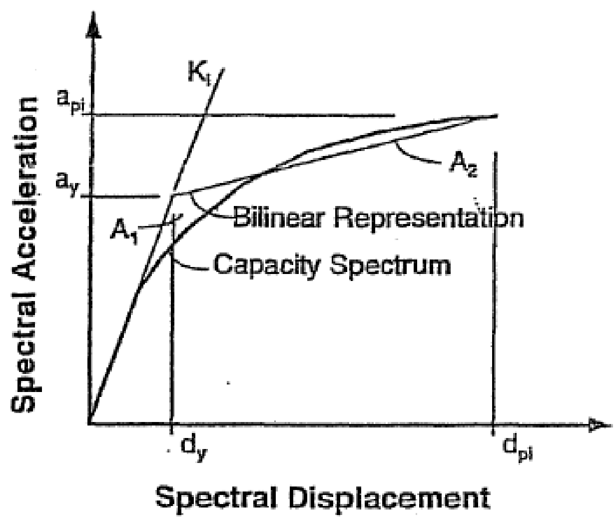


Figure 5.7: Bilinear representation of a capacity spectrum

5.2.4.3 Estimation of Damping and Reduction of 5 percent Damped Response Spectrum

The damping that occurs when earthquake ground motion drives a structure into the inelastic range can be viewed as a combination of viscous damping (inherent in the building) and hysteretic damping. Hysteretic damping is related to the area inside the loops that are formed when the earthquake force (base shear) is plotted and the structure displacement. It can be represented as equivalent viscous damping using equations that are available in the literature. The equivalent viscous damping, β_{eq}, connected with a maximum displacement of dpi, can be estimated from the following equation:

$$\beta_{eq} = \beta_0 + 0{,}05$$

Where,

- β_0: hysteretic damping represented as equivalent viscous damping
- 0.05: 5% viscous damping inherent in the structure (assumed to be constant)

The term $\beta_0$can be calculated as (Chopra 1995):

$$\beta_0 = \frac{E_D}{4\pi E_{SO}}$$

Where,

- E_D: energy dissipated by damping
- Eso: maximum strain energy

If the β_0 is written in terms of percent critical damping, the equation becomes (Figure 5.8):

$$\beta_0 = \frac{63{,}7\ (a_y d_{pi} - d_y a_{pi})}{a_{pi} d_{pi}}$$

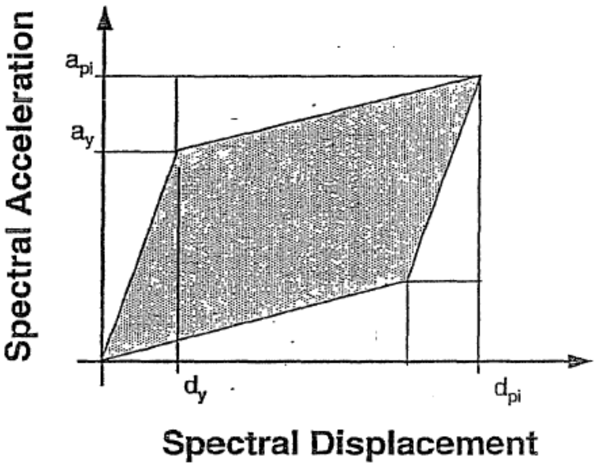


Figure5. 8: Hysteretic damping

The previous paragraphs referred to reinforced concrete buildings that are not typically ductile structures. In order to be consistent with the previously developed damping coefficients, as well as, to enable simulation of imperfect hysteresis loops (loops reduced in area), the concept of effective viscous damping using a damping modification factor, K, has been introduced.

Effective viscous damping:

$$\beta_{eff} = k\beta_0 + 5 = \frac{63{,}7\ \kappa\ (a_y d_{pi} - d_y a_{pi})}{a_{pi} d_{pi}} + 5$$

The κ-factor depends on the structural behavior of the building which in turn depends on the quality of seismic resisting system and the duration of ground shaking. ATC-40 defines three different structural behavior types. Type A represents hysteretic behavior with stable, reasonably full hysteresis loops while Type C represents poor hysteretic behavior with severely pinched and degraded loops. Type B denotes hysteresis behavior intermediate between Type A and Type C(Table 5.1).

Shaking Duration	Essentially New Building	Average Existing Building	Poor Existing Building
Short	Type A	Type B	Type C
Long	Type B	Type C	Type C

Table 5.1: Structural behavior types for the quality of seismic resisting system and the duration of ground shaking

The ranges and limits for the values of κ assigned to the three structural behavior types are given in Table 5.2.

Table 5.2 : Values for Damping Modification Factor, κ (ATC-40 [3])

Structural Behavior Type	**ζ_o (percent)**	**κ**
Type A	≤ 16.25	1.0
	> 16.25	$1.13 - \frac{0.51(Sa_y Sd_i - Sd_y Sa_i)}{(Sa_i Sd_i)}$
Type B	≤ 25	0.67
	> 25	$0.845 - \frac{0.446(Sa_y Sd_i - Sd_y Sa_i)}{(Sa_i Sd_i)}$
Type C	Any value	0.33

Table 5.2: Values for damping modification factor K

5.2.4.4 Numerical Derivation of Spectral Reductions

The equations for the reduction factors SRA and SRV are given by:

$$SR_A = \frac{1}{B_S} = \frac{3.21 - 0.68\ln\beta_{eff}}{2.12}$$

$$SR_V = \frac{1}{B_L} = \frac{2.31 - 0.41\ln\beta_{eff}}{1.65}$$

The values for *SRA* and *SRV* should be greater than or equal to the values given in table below:

Structural behavior type	SR_A	SR_V
Type A	0.33	0.50
Type A	0.44	0.56
Type A	0.56	0.67

Table 5.3: Minimum values of damping reduction factors

5.2.4.5 Intersection of Capacity Spectrum and Demand Spectrum

When the displacement at the intersection of the demand spectrum and the capacity spectrum di, is inside $0.95a_{pt} \leq di \leq 1.05d_{pt}$ of the displacement of the trial performance point, the $a_{pt}d_{pt}$, d_{pt} becomes the performance point. If the meeting point of the demand spectrum and the capacity spectrum is not within the acceptable tolerance, then a new a_{pt}, d_{pt} point is selected and the process is repeated. Figure beneath (Figure 5.9) illustrates the theory. The performance point represents the maximum structural displacement expected for the demand earthquake ground motion.

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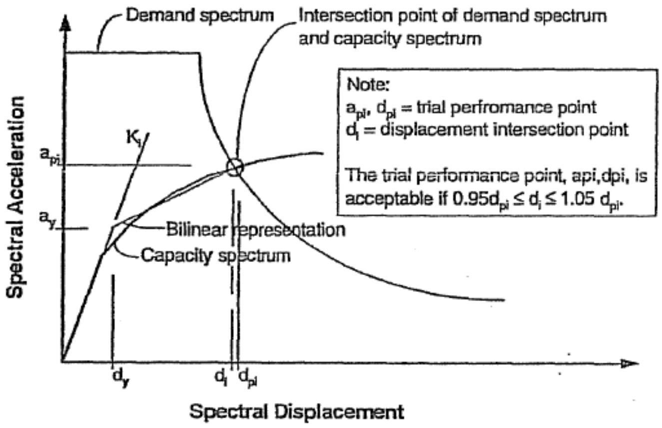

Figure 5.9: intersection of the demand spectrum and the capacity spectrum (performance point)

5.2.4.6 Performance point of the structure

After the convergence of the values the spectral displacement demand is converted to global (roof) displacement by multiplying estimated spectral displacement demand of equivalent SDOF system with first modal participation factor at the roof level.

5.3 Summary of pushover analysis

Pushover analysis yields insight into elastic and inelastic response of construction under earthquakes that provided the adequate modeling of structure. However, pushover analysis is more appropriate for low to mid-rise buildings with dominant fundamental mode response. For special and high-rise buildings, pushover analysis should be complemented with other evaluation procedures since higher modes could certainly affect the response. For implement the pushover analysis the process is described below.

Initially plastic hinges are determining the properties of them. Next the plastic hinges assign to members the plastic hinges of the construction. Subsequently is determining the length of the plastic hinge zones. After that the first load step of construction comprising is verified, imposing vertical dead and live loads in the same time (the initial conditions). Afterward the horizontal seismic loads are defined (the second step of the loading). The horizontal seismic loads which were selected are the lateral forces Fi. These forces are determined by distributing, along the height of the building, the base shear force. In this step, it becomes the selection of movement control.

Finally the analysis is performed. The lateral load is gradually increased until the construction to develop maximum roof displacement equal to the displacement control which is chosen. Then export of results related to developing cutting base and roof movement for each step of the analysis to obtain the base shear diagram – displacement of the node (Pushover curve).

5.3.1 Loads for pushover analysis

The pushover analysis is done in two phases. Initially dead and live loads are imposed on the structure which are the initial conditions, before the loading of the construction with horizontal lateral loads. The dead and live loads are combined with non-linear analysis.

In this part of the thesis the horizontal seismic loads are defined. The horizontal seismic load which selected is the lateral forces Fi. These forces are determined by distributing along the height of the structure the base shear force, which is determined according to the following formula:

$$F_b = S_d\left(T_1\right) m \lambda$$

Where:

- S d(T1) is the design spectral acceleration corresponding to the fundamental period of vibration T1,
- m is the total mass of the building

- λ is a correction factor.

In the simplified variant, lateral forces Fi are determined according to the following expression:

$$F_i = F_b \frac{m_i z_i}{\sum_{i=1}^{N} m_i z_i}$$

Where

- Mi is the mass of storey i,
- And zi is the height of storey i with respect to the base of the structure.

The next diagram (Figure 5.10) illustrates the design spectrum of EC8 and the design spectral acceleration corresponding to the fundamental period of vibration. For the X-direction, the fundamental period is equal to Tx = 1.45 sec and the corresponding acceleration for the spectrum of EC-8 is: Sa (Tx) = 0.8126 m/s^2. For the Y-direction, the fundamental period is equal toTy = 0.814 sec and the corresponding acceleration for the spectrum of EC8 is: Sa (Ty) = 1.317 m/s^2.

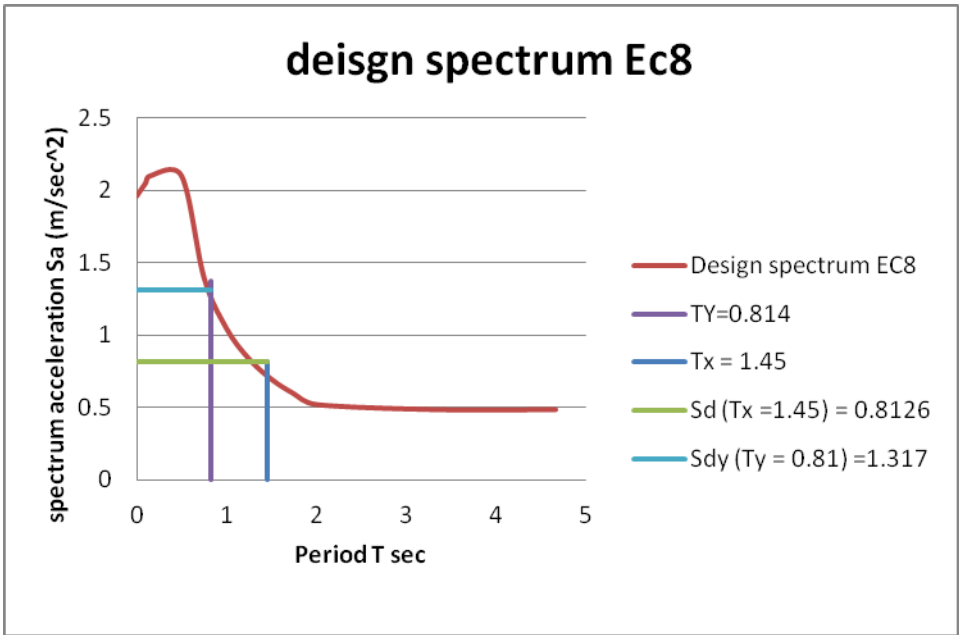

Figure 5.10: The design spectrum of EC8 and the design spectral acceleration

In Table 5.4 can be seen the way the mass is distributed along the height of the structure, the level of each floor and the m * zi.

LEVEL	MASS KN	MASS kg	z (m)	m * z
ground floor	3250	331.29	2.7	894.4
floor 1	3163	322.42	5.7	1837.8
floor 2	3163	322.42	8.7	2805.1
floor 3	3163	322.42	11.7	3772.38
floor 4	3163	322.42	14.7	4739.6
floor 5	1855	189	17.7	3346.9
top floor	1484	151.2	20.7	3131.3
total mass		**1961.3**	**Σ(mi*zi)**	**20527.8**

Table 5.4: The mass distribution along the floors

- To calculate the base shear force, is used the formula:

$$F_b = S_d(T_1)m\lambda$$

- The base shear force in each direction is presented below:

Fbx (KN)	**1355**
Fby (KN)	**2196**

- The lateral forces Fi are computed according to the subsequent expression:

$$F_i = F_b \frac{m_i z_i}{\sum_{i=1}^{N} m_i z_i}$$

Next table (Table 5.5) presents the lateral forces distributed along the height of the building:

LEVEL	Fx (KN)	Fy (KN)
ground floor	59.03	95.67
floor 1	121.2	196.57
floor 2	185.12	300.03
floor 3	248.95	403.49
floor 4	312.7	506.95
floor 5	220.88	357.98
top floor	206.65	334.92

Table5. 5: The lateral forces Fi in each level

5.4 Modeling pushover analysis

5.4.1 Plastic hinges

Plastic hinges are created at the ends of the beams and at the base of the columns which lead to the foundation. First the characteristics of plastic hinges have to assign for each section. A crucial force for the beams a critical for failure is the moment in the local axis 3 (M3). A big moment is possible to direct to failure. At the columns, a critical for failure, is the interaction between the axial force P and the moment in local axes 2 and 3 (M2-M3). The characteristics for plastic hinges are given at both ends of the element (relative distance 0 and 1).

- Select all the beams

  Assign → Frame → Hinges → add (relative distance 0 and relative distance 1)

  Degree of freedom M3

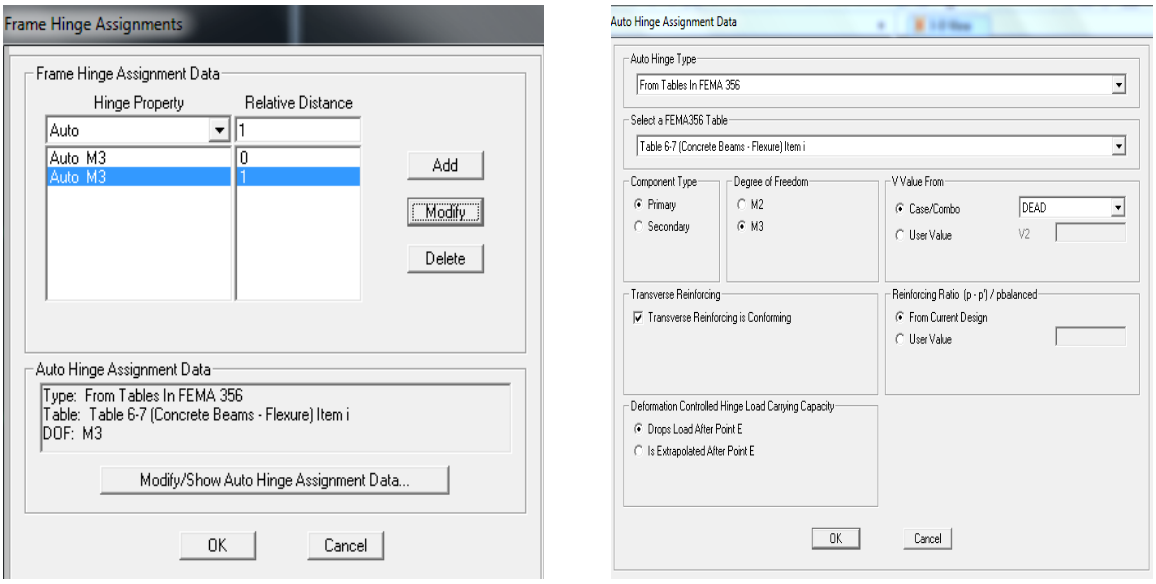

Figure 5.11: Assign plastic hinges for the beams

- The same procedure is repeated for the columns.

  Select all the columns

  Assign → Frame → Hinges → add (relative distance 0 and relative distance 1)

  Degree of freedom P-M2- M3

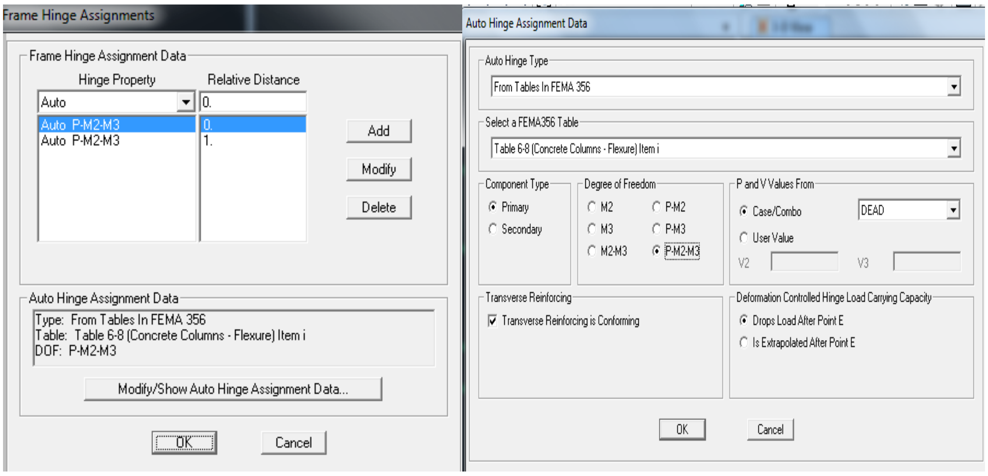

Figure 5.12: Assign plastic hinges for the columns

- The behavioral characteristics of the plastic hinges which are selected in accordance with Regulation Fema 356 are be made understandable, with the following procedure :

  Define → Section properties → Hinges properties → Shoe generated props (Modify/show property)

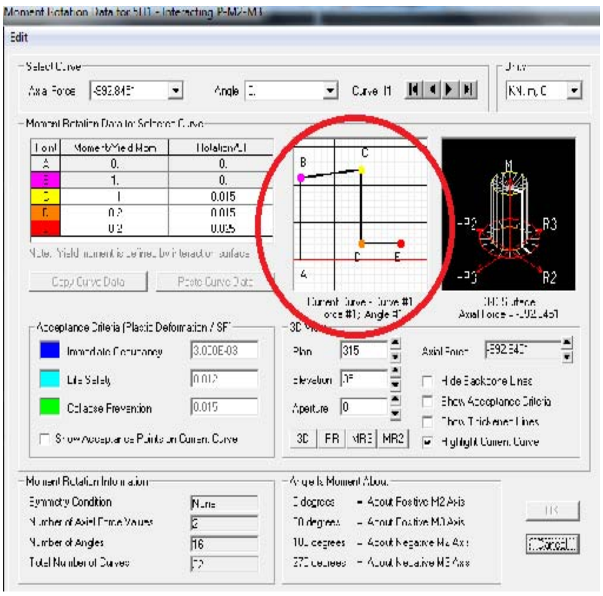

Figure 5.13: The behavioral characteristics of the plastic hinges

5.4.2 Criteria failure for the materials

The next step for the pushover method is to identify the criteria of failure for the materials. For the concrete the compression zone is set at 0.0035 and for the steel is equal to 0.02. These are introduced in the program by selecting:

- Define → Materials → Modify/show material→ Modify/show material properties → Advanced material Property Data properties → Nonlinear material data (change the strain capacity)

For concrete:

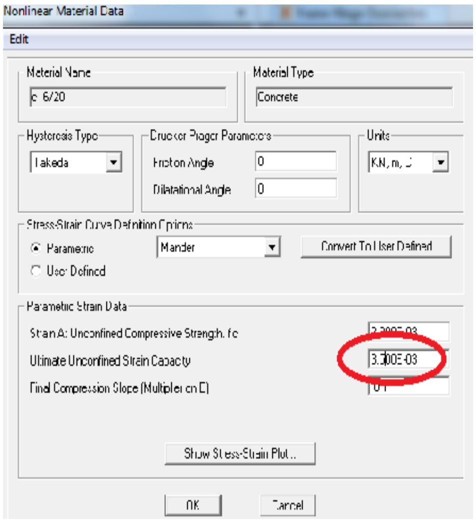

Figure 5.14: Criteria of failure for concrete

For steel:

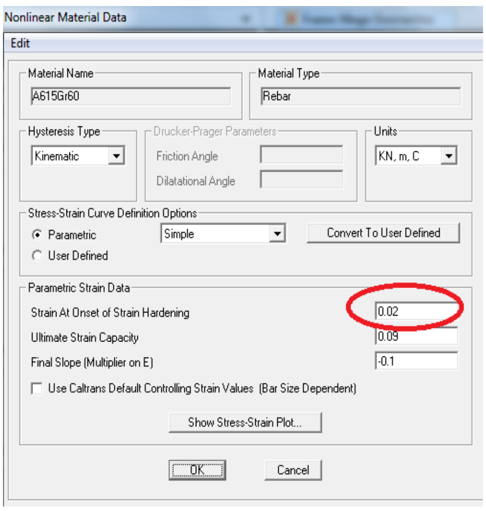

Figure 5.15: Criteria of failure for steel

5.4.3 Loads

The pushover analysis is completed in two phases. Firstly dead and live loads are imposed on the building (the initial conditions). After that the loading of the construction with horizontal lateral loads is made.

5.4.3.1 First phase of pushover

First of all the initial condition for the analysis is created .The loading of pre-existing lateral loads are dead and live vertical loads. To provide initial conditions should define Load Case containing dead and live load. The type of analysis for the vertical loads is non-linear. This is introduced by selecting:

- Define → Load case → Add new load case → name (G +0.3Q non lin) and is selected analysis type non linear .

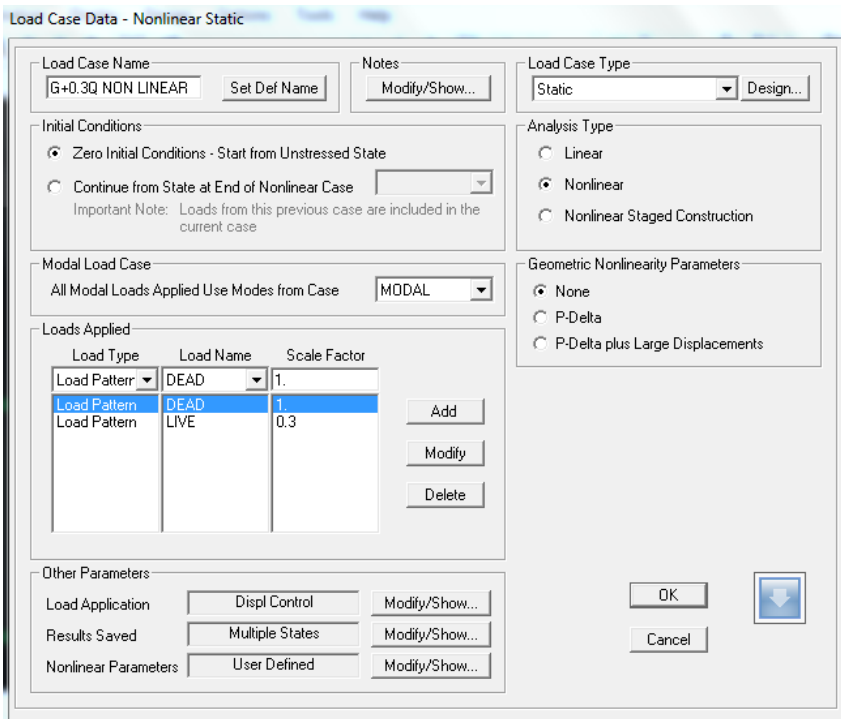

Figure 5.16: Introduction in the program the load combination for the first phase of the pushover analysis

5.4.3.2Modeling the seismic horizontal loads

The horizontal seismic loads are defined which are the second step of the loading. The horizontal seismic load which selected is the lateral forces Fi. A new load pattern is created with name lateral force. The same formula is kept for the y direction (LATERAL Y). This is launched in the program by choosing:

- Define → Load pattern name LATERAL X and LATERAL Y → type (OTHER) → Add new load pattern

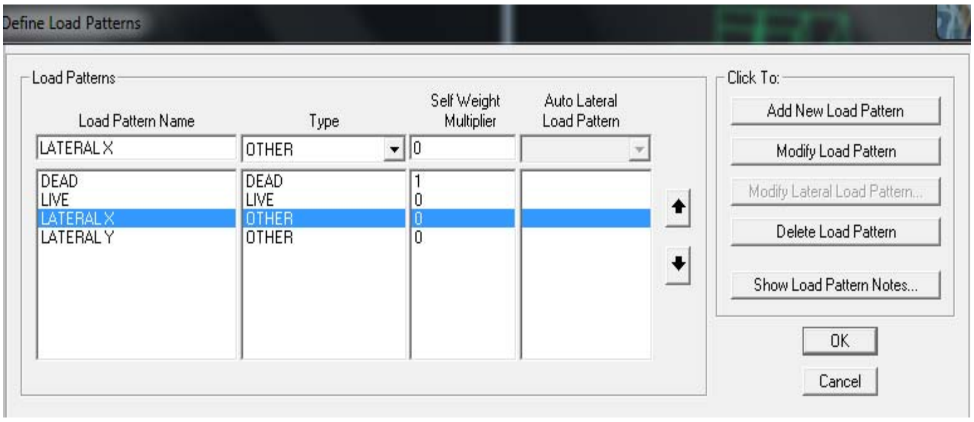

Figure 5.17: New load pattern for lateral forces

Lateral forces were applied at the nodes on one side of the structure in both directions. The procedure can be seen below :

- Select the suitable nodes

Assign → Joint → Forces → load pattern name (LATERAL X or LATERAL Y)

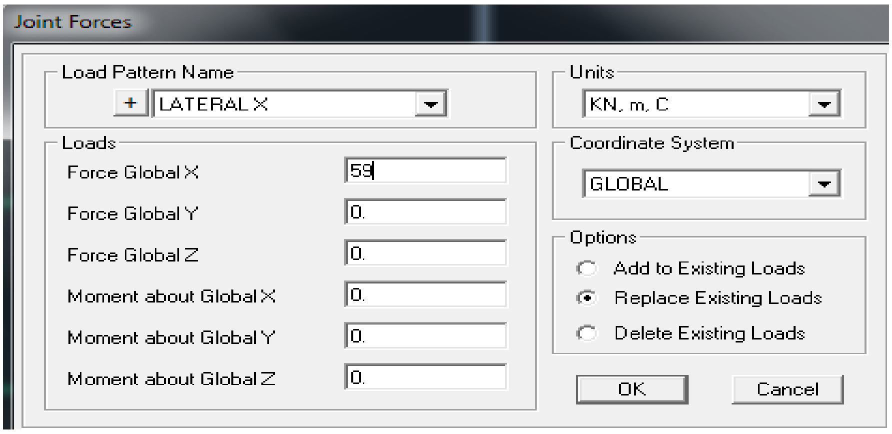

Figure 5.18: Applying the lateral forces at the nodes

The figures underneath show the distribution of the forces at the two directions (X – Y) (Figures 5.19 & 5.20).

Direction X:

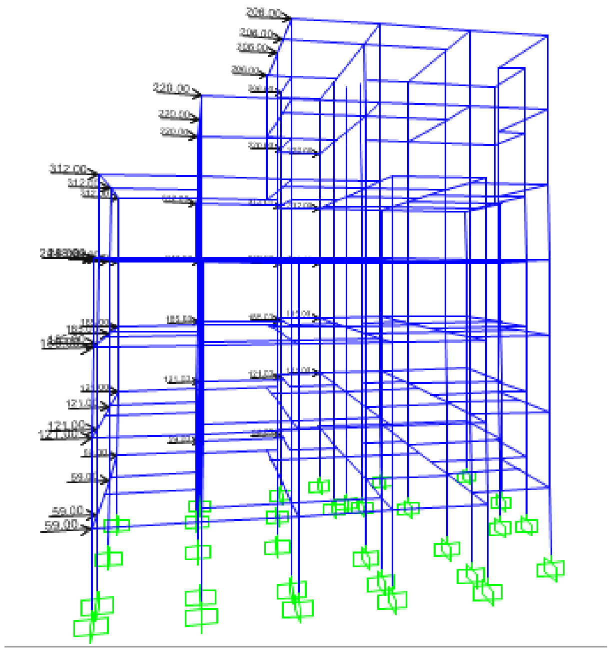

Figure 5.19: Distribution of the lateral forces at x direction

Direction Y:

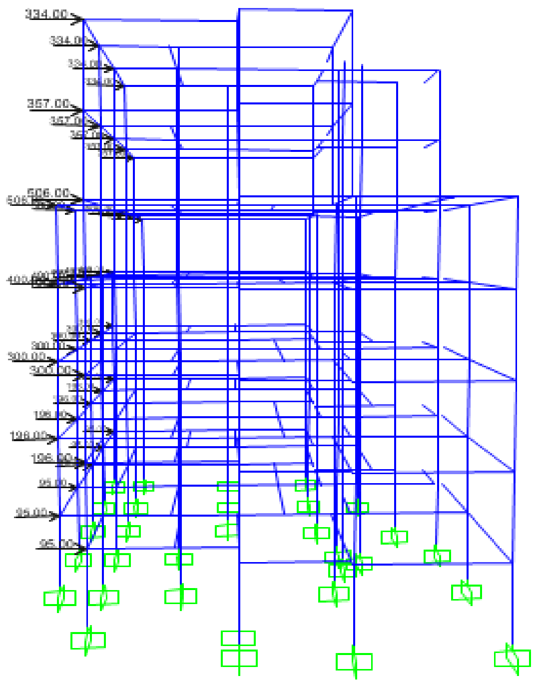

Figure 5.20: Distribution of the lateral forces at y direction

5.4.4 Load case for Pushover analysis

At this point the load combination is defined. The new load case which is defined has as initial conditions the vertical loads (dead and live loads) and for the second phase the horizontal loads (lateral load). The commands are:

Define → Load case → Add new load case

- name (PUSH +X)
- Load case type : static
- initial conditions : Continue from state at end of nonlinear case → G+0.3Q non lin
- Load name : LATERAL x
- Analysis type :Nonlinear
- Geometry Nonlinearity Parameters :None

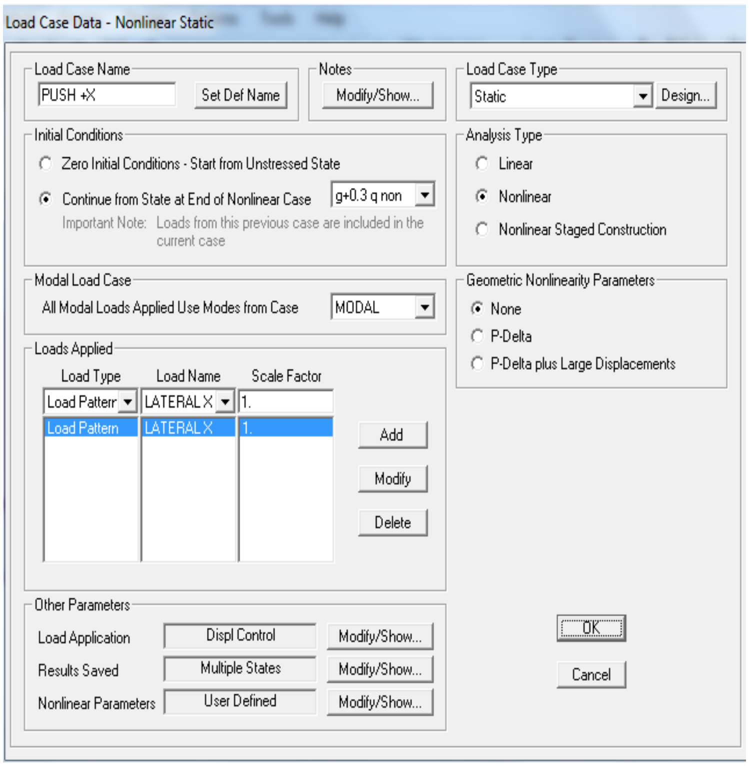

Figure 5.21: Introduction in the program the load case (PUSH +X) for pushover analysis

The way to control the pushover analysis is the displacement of the master node. The horizontal load sets the value of the lateral load which will begin pushover analysis while gives the ratio between the horizontal loads of stories that will remain constant during their increase. The master node is needed to define and the displacement for which the application of lateral load would stop and pushover analysis will finish. In this case the master node is 459 and the equivalent displacement is 0.5m and 0.2m for x and y direction respectively. The procedure in the program is:

Define → Load case → Select the load case (PUSH +X) → other parameters →Load application →Modify /show

Then master node is defined by selecting the below:

- Load application control : Displacement control
- Control displacement : Use monitored displacement
- Load a monitored displacement magnitude of → 0.50
- DDF : UI
- Joint : 449

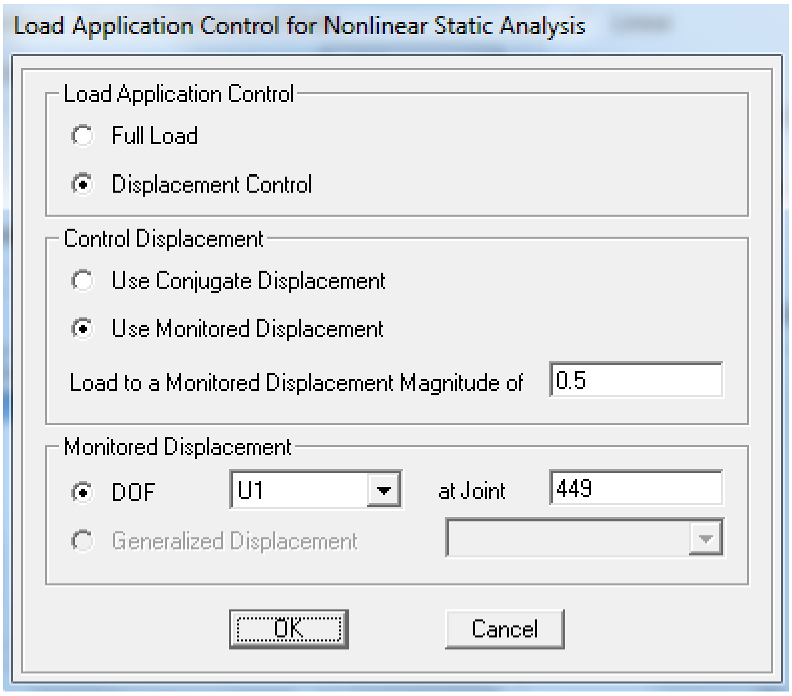

Figure 5.22: Load application control for non linear static analysis

Sometimes, problems are occurring due to the convergence of numerical methods. These problems lead to not get the desired results from the method. To avoid that usually is needed to increase the beneath numbers:

Define → Load case → Select the load case (PUSH +X) → Other parameters →Non Linear parameters →Modify /show

In the Figure 5.23 the numbers were changed can be observed:

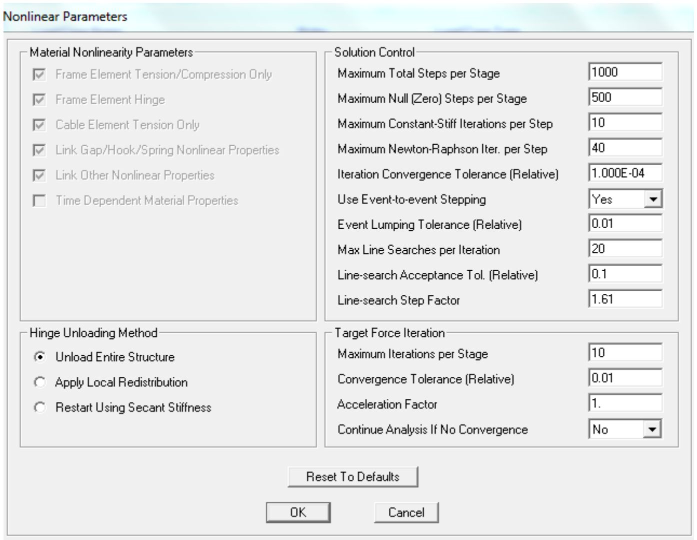

Figure 5.23: The non linear parameters that must be changed

The numbers which are used at this case study are:

- Maximum Total Steps per Stage: 1000
- Maximum Null Steps per Stage: 500

Pushover analysis is carried out separately in the X and Y directions. The table underneath (Table 5.6) illustrates the four different load cases for non linear static analysis:

Load case name	**Load name**	**Scale Factor**
PUSH +X	LATERAL X	1
PUSH -X	LATERAL X	-1
PUSH +Y	LATERAL Y	1
PUSH -Y	LATERAL Y	-1

\Table5. 6: The load cases for the pushover analysis

5.5 Static pushover analysis results

Introduction

The pushover analysis of a structure is a static non-linear analysis under permanent vertical loads and gradually increasing lateral loads. The equivalent static lateral loads approximately represent the intensity of earthquake included forces. A plot of the total base shear with the top displacement in a construction is obtained by this process. That would indicate to premature failure or weakness. The analysis is carried out up to failure, thus it enables determination of collapse load and ductility capacity.

5.5.1 Base shear versus top displacement

A static nonlinear (pushover) analysis of the building was carried out using SAP 2000. A maximum roof displacement of 0.50m was chosen to be applied. Pushover analysis was carried out separately in the X and Y directions. The resulting pushover curves, in terms of Base Shear – Roof Displacement (V-Δ), are given in figures below (Figures 5.24, 5.25, 5.26 & 5.27) for X and Y directions respectively. The slope of the pushover curves is gradually changed with increase of the lateral displacement of the building. This is due to the progressive formation of plastic hinges in beams and columns throughout the structure. The pushover curves reach a maximum which corresponds to failure of the building. There are many plastic hinges formed with big plastic rotations and the building can no longer sustain them. The curves were calculated and plotted for displacement more than the minimum required, 150% of the targeted, but not designed by the essential drop in strength of the structure because it presents instabilities in the algorithm. Program generally cannot make the curve until the fall of the strength, so there is not a sudden drop of the curve.

<u>Direction +X:</u>

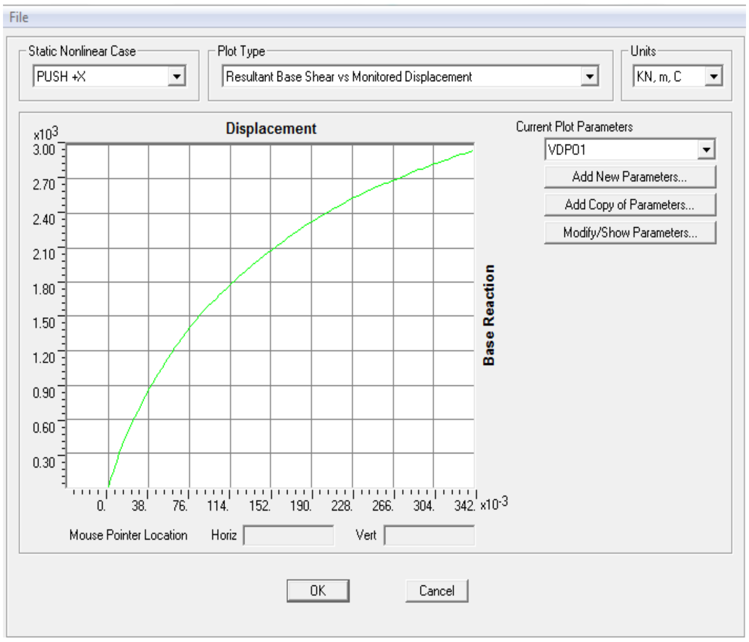

Figure 5.24: The pushover curve, in terms of Base Shear – Roof Displacement (V-Δ) for direction +X

<u>Direction -X:</u>

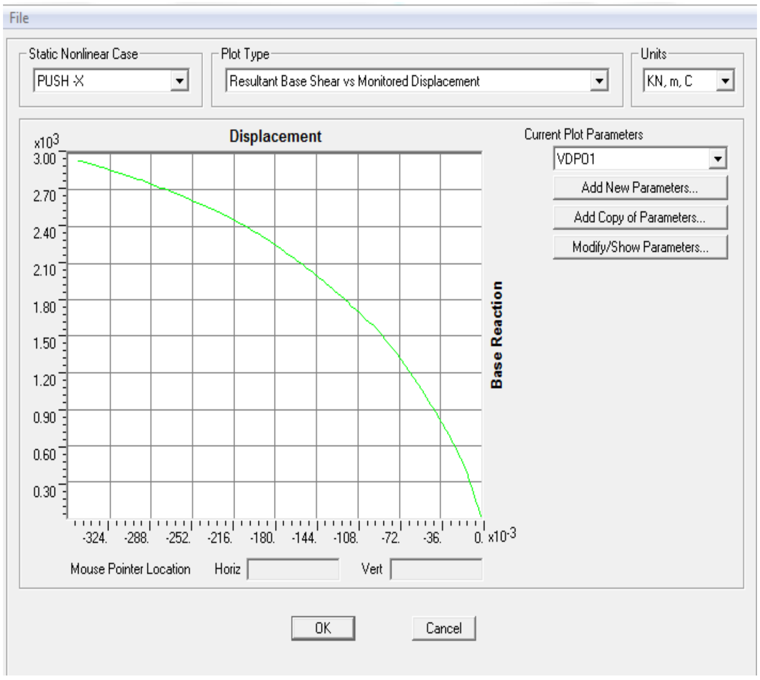

Figure 5.25: The pushover curve, in terms of Base Shear – Roof Displacement (V-Δ) for direction -X

Direction +Y **:**

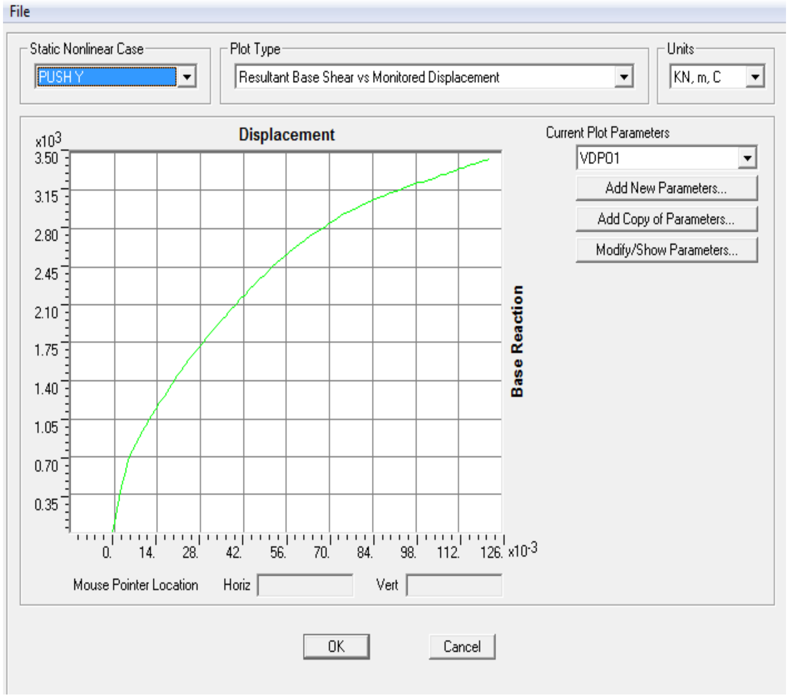

Figure 5.26: The pushover curve, in terms of Base Shear – Roof Displacement (V-Δ) for direction +Y

Direction –Y:

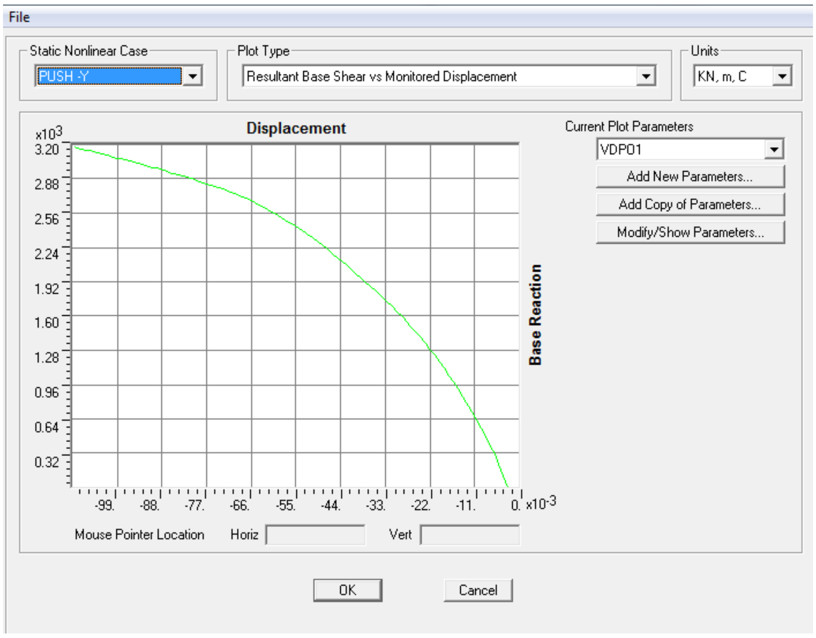

Figure 5.27: The pushover curve, in terms of Base Shear – Roof Displacement (V-Δ) for direction -Y

Figure 5.28 summarized the pushover curves, in terms of Base Shear – Roof Displacement (V-Δ) for all directions:

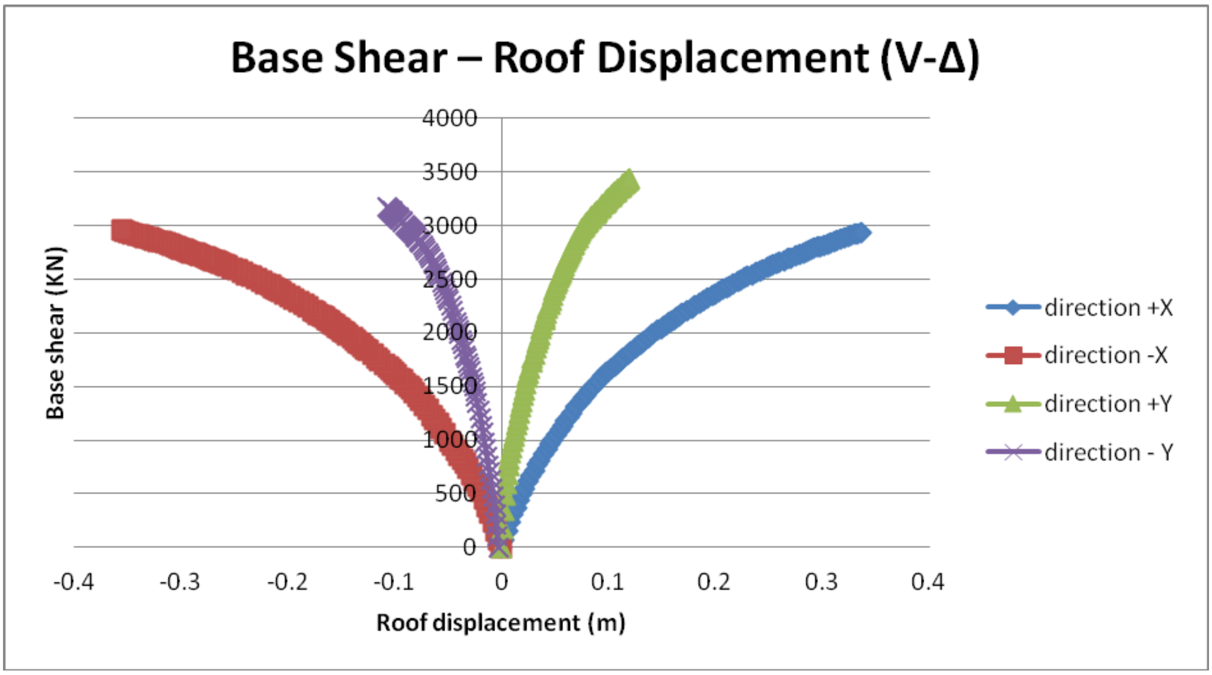

Figure 5.28: The pushover curves, in terms of Base Shear – Roof Displacement for all directions

The results from the curves in the X and – X directions are similar. The maximum point for +X direction is V =2959 kN and Δ = 0.34 m. For -X direction V = 2928 kN and Δ = -0.35m, with the -X pushover curve being slightly worse than the +X curve.

Comparing the pushover curves in the +Y and – Y directions is detected that the results have not great difference. The max point for +Y direction is V = 3452kN and Δ = 0.12m and for -Y direction is V = 3189kN and Δ = -0.109 m.

The +Y pushover curve is being stiffer than the -X curve .This is explained by the fact that in the +Y direction the walls (the length of the walls at the lift is 1.50m) are exist. The building at +Y direction receives more forces with smaller displacement. Therefore the structure in +Y direction has more stiffness as compared to +X direction which is more flexible

5.5.2 Performance point

The performance point is the point where the capacity curve crosses the demand curve according to ATC-40. At the next figures show the performance point for all push over curves .With red color is the elastic spectrum of EC8 and a series of reduced responses to ADRS format. The green curve represents the spectrum resistance of equivalent SDOF system as shown by the resistance curve. With the orange line defined as the locus of points as defined by ATC-40. The intersection of the orange line (demand) and the green curve (capacity) is the performance point.

Direction +X: This analysis was completed in 107 steps and performance point was set between steps 60 and 61 of the analysis .The performance point Sd is equal to 0.151 m.

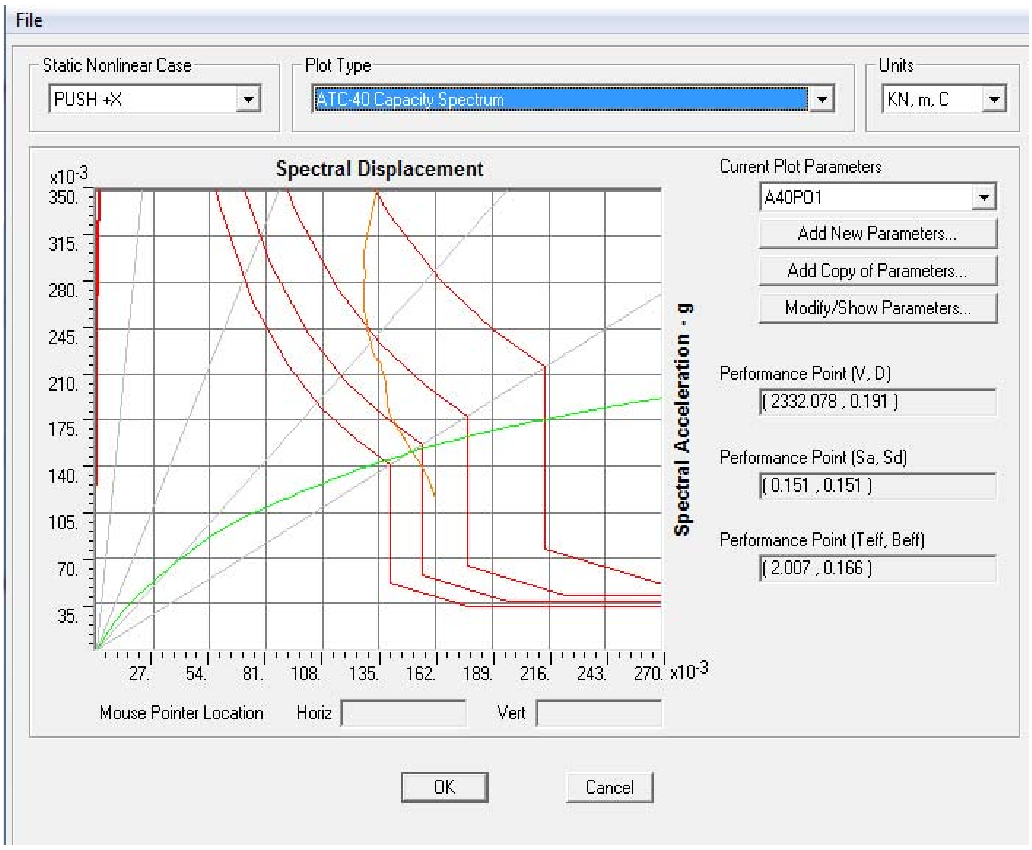

Figure 5.29: Pushover curve for +X direction

The table below (Table 5.7) displays some of the steps of the analysis and for each step shows the details for the capacity and demand curve for the equivalent one degree of freedom system.

Direction +X								
Capacity curve			Capacity and demand curve					
step	diplacement (m)	Base force KN	Teff	Beff	SdCapacity(m)	SaCapacity	SdDemand(m)	SaDemand
58	0.169503	2410.209	1.985446	0.164568	0.144586	0.147656	0.149921	0.153103
59	0.174205	2443.933	1.995744	0.165385	0.147629	0.149211	0.150341	0.151952
60	0.191153	2332.06	2.006709	0.166366	0.150826	0.150781	0.150828	0.150783
61	0.180877	2491.249	2.018488	0.167401	0.154304	0.152462	0.151405	0.149598
62	0.183389	2509.323	2.028444	0.168373	0.157202	0.153805	0.151852	0.148571
63	0.185901	2527.381	2.039182	0.169475	0.160313	0.155201	0.152308	0.147452

Table5. 7: Computation of performance point for +X direction

The figure below (Figure 5.30) presents the overall yielding pattern of the structure at the performance point.

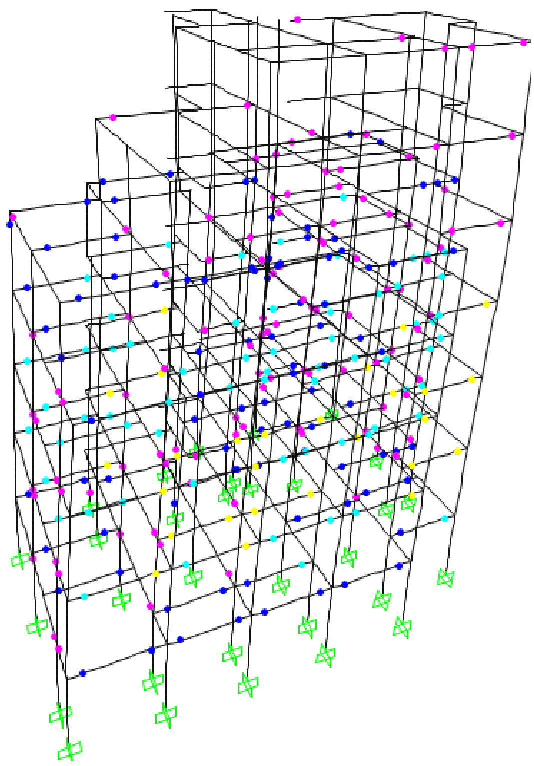

Figure 5.30: The yielding pattern of the structure at the performance point for +X direction

In the table below (Table 5.8) the elements which has entered in the plastic zone is shown.

Step	Displacement (m)	BaseForce KN	AtoB	BtoIO	IOtoLS	LStoCP	CPtoC	CtoD	DtoE	BeyondE
58	0.183263	2283.969	632	110	102	69	0	9	0	0
59	0.187104	2307.689	631	109	99	67	0	16	0	0
60	0.191153	2332.06	631	108	91	65	0	27	0	0
61	0.195547	2357.929	631	106	90	58	0	37	0	0
62	0.199212	2378.601	627	109	82	56	0	48	0	0
63	0.203135	2399.978	625	110	76	51	0	60	0	0

Table 5.8: Computed limit states for the studied building for + X direction

From the above can be concluded that most elements have entered in the plastic zone (blue color). Twenty-seven elements of the building are in the Collapse (CD) limit state. This means that the building requires retrofitting. Furthermore some of the beams have surpassed 75 % of limit for the chord rotation. Consequently, for the purpose of design <<LIFE SAFETY >> the original operator is not sufficient for this charging.

From the Figure 5.30 it can be summarized that the beams which are embedment in the plate are not adequate. These beams require retrofitting. The figure below (Figure 5.31**)** shows the diagram of moment- plastic rotation for the beam K1.7.

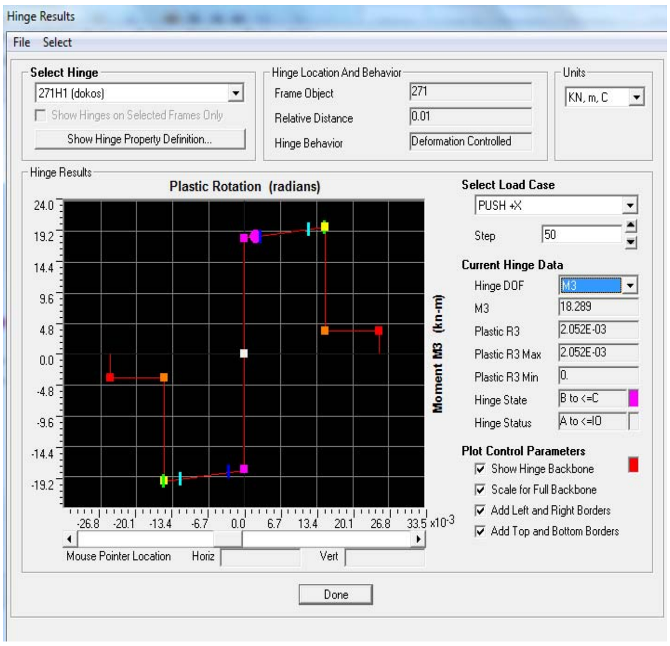

Figure 5.31: Moment - rotation curve

<u>Direction -X:</u> This analysis was completed in 111 steps and performance point set between steps 60 and 61 of the analysis .The performance point Sd is 0.152 m .

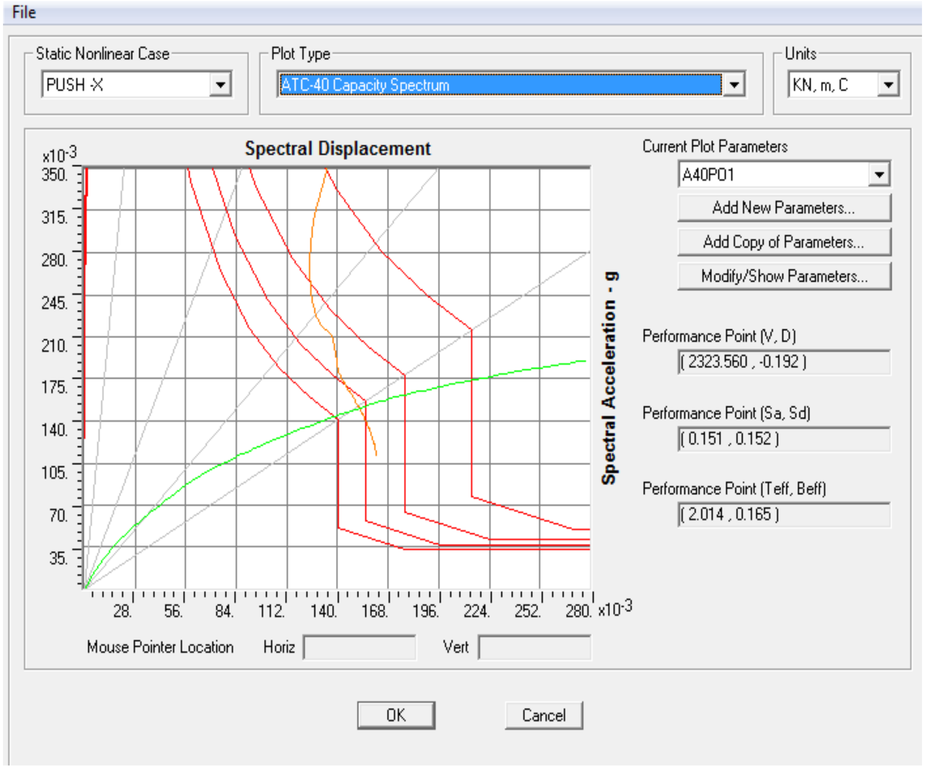

Figure 5.32 : Pushover curve for -X direction

The table (Table 5.9) illustrates some of the steps of the analysis. For each one it can be observed the details for the capacity and the demand curve for the correspondent one degree of freedom system.

Direction -X								
Capacity curve			Capacity and demand curve					
step	diplacement (m)	Base force KN	Teff	Beff	SdCapacity(m)	SaCapacity	SdDemand(m)	SaDemand
58	-0.183945	2273.255	1.992185	0.162728	0.145486	0.147571	0.150966	0.15313
59	-0.186454	2289.149	1.998906	0.163298	0.147472	0.148582	0.151224	0.152362
60	-0.190154	2311.972	2.008879	0.164211	0.150397	0.150028	0.151694	0.151321
61	-0.192663	2326.938	2.015712	0.164881	0.152384	0.150981	0.152006	0.150606
62	-0.19657	2350.11	2.02623	0.165889	0.155476	0.152449	0.152486	0.149518
63	-0.200641	2373.488	2.037238	0.166984	0.158707	0.15394	0.152966	0.148371

Table5. 9: *Computation of performance point for -X direction*

The next picture (Figure 5.33) displays the overall yielding pattern of the construction at the performance point.

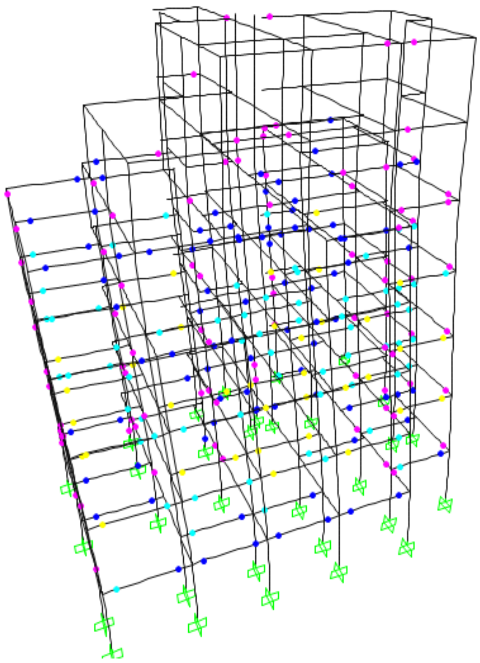

Figure 5.33: The yielding pattern of the structure at the performance point for –X direction

Afterwards the table (Table 5.10) with the number of element which has entered in the plastic zone is shown.

Step	Displacement (m)	BaseForce KN	AtoB	BtoIO	IOtoLS	LStoCP	CPtoC	CtoD	DtoE	BeyondE
58	-0.183945	2273.255	648	90	101	62	0	21	0	0
59	-0.186454	2289.149	647	90	96	62	0	27	0	0
60	-0.190154	2311.972	646	89	91	62	0	34	0	0
61	-0.192663	2326.938	645	89	88	64	0	36	0	0
62	-0.19657	2350.11	639	93	82	64	0	44	0	0
63	-0.200641	2373.488	638	93	78	62	0	51	0	0

Table 5.10: Computed limit states for the studied building for - X direction

Thirty six elements of the building are in the Collapse (CD) limit state, which means that the building requires retrofitting. It is observed that the structural elements of the two upper floors have not entered in the plastic zone in contrast to some structural elements in the lower floors, which already have collapsed (yellow color). The elements which are entered in the plastic area can be compared with more accuracy among the previous analysis

<u>Direction +Y:</u> This analysis was completed in 97 steps and performance point set between steps 43 and 44 of the analysis .The performance point Sd is 0.156 m.

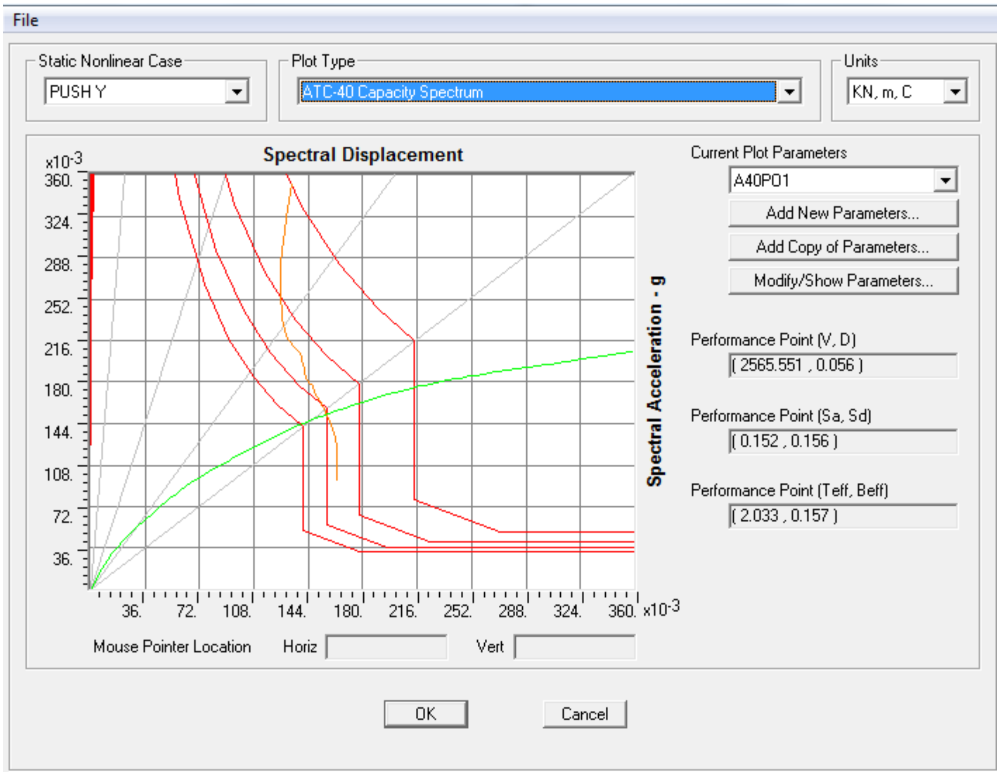

Figure 5.34 : Pushover curve for +y direction

The (Table 5.11) presents some of the steps of the method. For each step the details for the capacity and demand curve are given for the equivalent one degree of freedom system

Direction +Y								
Capacity curve			**Capacity and demand curve**					
step	**diplacement (m)**	**Base force KN**	**Teff**	**Beff**	**SdCapacity(m)**	**SaCapacity**	**SdDemand(m)**	**SaDemand**
40	0.051553	2451.642	1.984369	0.151984	0.141319	0.144476	0.154056	0.157497
41	0.052619	2479.726	1.995489	0.153065	0.14469	0.146278	0.154439	0.156134
42	0.053836	2511.26	2.008227	0.154307	0.148586	0.148317	0.15497	0.15469
43	0.055212	2543.269	2.022733	0.155985	0.15287	0.150413	0.155535	0.153035
44	0.056444	2571.637	2.035753	0.157459	0.156766	0.15228	0.156038	0.151572
45	0.057934	2605.131	2.051101	0.159086	0.161476	0.154516	0.156651	0.149899

Table 5.11: Computation of performance point for +Y direction

In the following image (Figure 5.35) can be seen the overall yielding pattern of the building at the performance point.

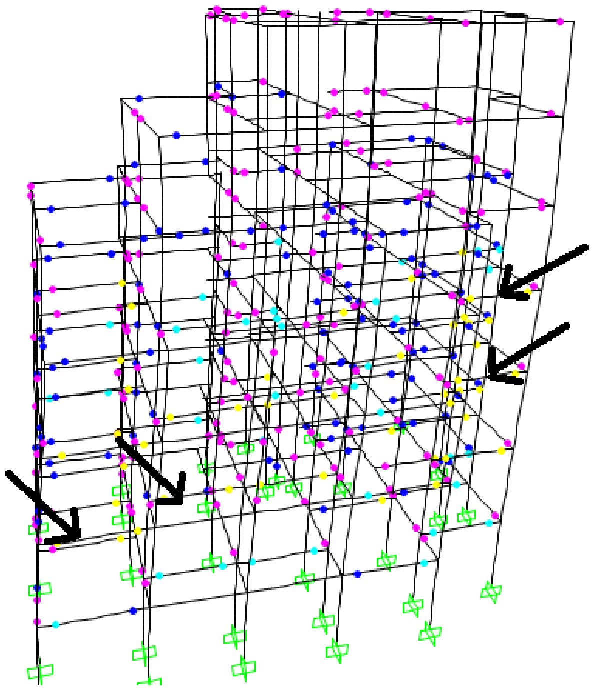

Figure 5.35: The yielding pattern of the structure at the performance point for +Y direction

The table below (Table 5.12) shows the elements were entered in the plastic zone.

Step	Displacement (m)	BaseForce KN	AtoB	BtoIO	IOtoLS	LStoCP	CPtoC	CtoD	DtoE	BeyondE
40	0.051553	2451.642	594	103	143	46	0	36	0	0
41	0.052619	2479.726	590	103	145	45	0	39	0	0
42	0.053836	2511.26	586	102	142	44	0	48	0	0
43	0.055212	2543.269	582	106	134	45	0	55	0	0
44	0.056444	2571.637	579	105	130	48	0	60	0	0
45	0.057934	2605.131	574	107	130	42	0	69	0	0

Table 5.12: Computed limit states for the studied building for +Y direction

From the above it is observed that large number of plastic hinges have created with focusing at the beams. In addition 60 elements have exceeded the limit level of <<Life safety >>. These elements have entered in the Collapse (CD) limit state. On the other hand the columns are not entered in plastic zone. Consequently, for the purpose of design <<LIFE SAFETY >> the original operator is not sufficient for this charging.

Direction -Y: This analysis was completed in 91steps and the performance point set between steps 46 and 47 of the analysis .The performance point Sd is equal with 0.160m.

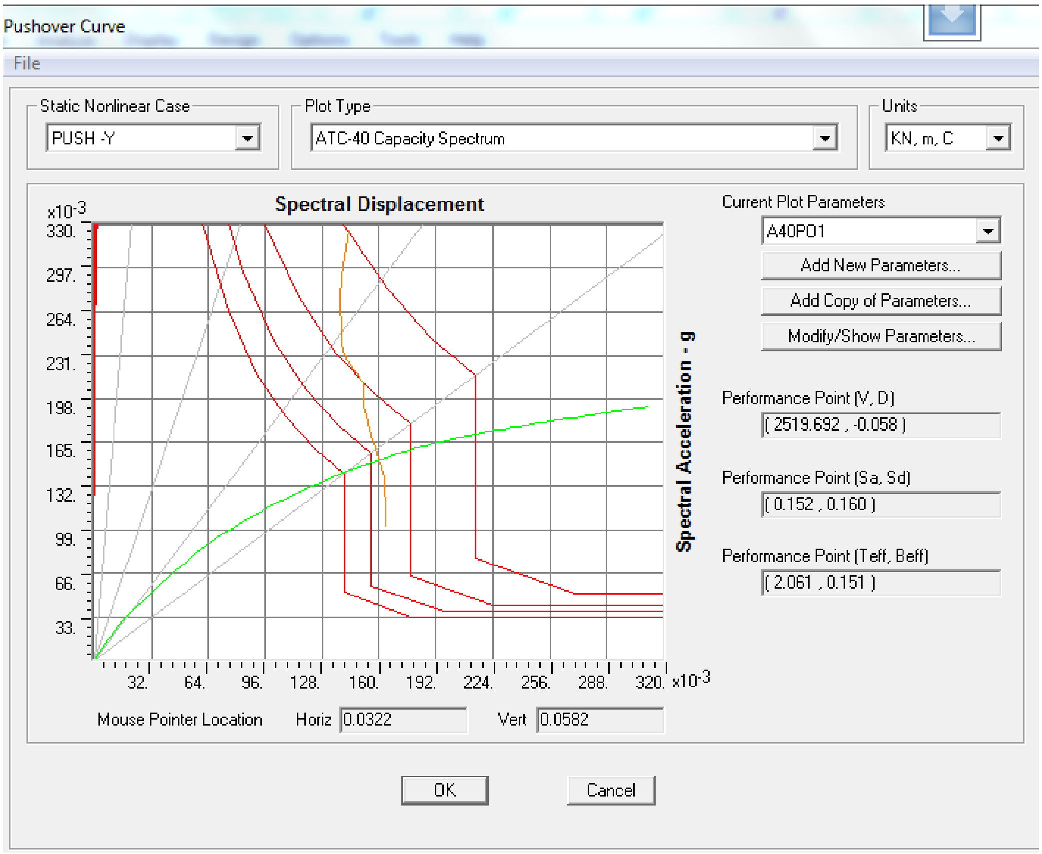

Figure 5.36 : Capacity curve for –Y direction

In the subsequent table (Table 5.13) can be noted some of the steps from the performance. For each step the details for the capacity and the demand curve are given for the specific one degree of freedom system.

Direction -Y								
Capacity curve			**Capacity and demand curve**					
step	**diplacement (m)**	**Base force KN**	**Teff**	**Beff**	**SdCapacity(m)**	**SaCapacity**	**SdDemand(m)**	**SaDemand**
44	-0.055168	2436.292	2.01781	0.145885	0.147847	0.146181	0.158746	0.156957
45	-0.056172	2461.18	2.028548	0.147258	0.151083	0.147803	0.1591	0.155646
46	-0.057601	2495.452	2.044006	0.149324	0.15569	0.150016	0.15956	0.153744
47	-0.059481	2539.379	2.063755	0.151799	0.161738	0.152874	0.16018	0.151402
48	-0.060503	2562.457	2.074576	0.153187	0.165045	0.154377	0.160494	0.15012
49	-0.061935	2594.669	2.089462	0.154978	0.169707	0.156484	0.160952	0.148411

Table 5.13: Computation of performance point for –Y direction

The figure below (Figure 5.37) displays the overall yielding pattern of the construction at the performance point.

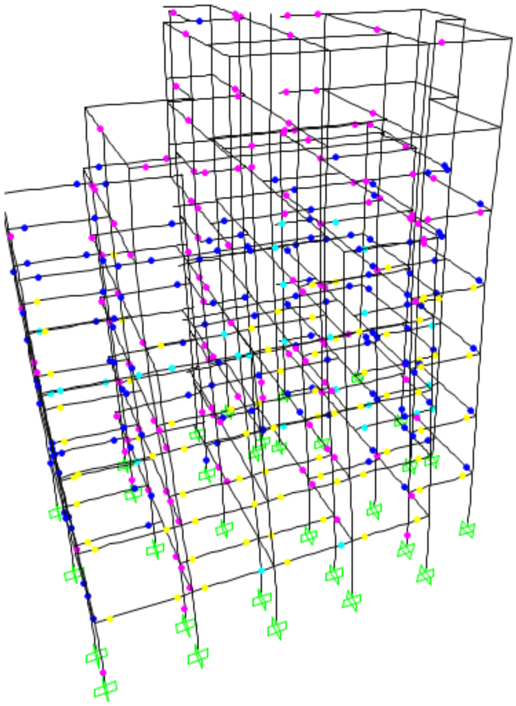

Figure 5.37: The yielding pattern of the structure at the performance point for –Y direction

The table 5.14 can be observed the number of elements entered the plastic zone.

Step	Displacement (m)	BaseForce KN	AtoB	BtoIO	IOtoLS	LStoCP	CPtoC	CtoD	DtoE	BeyondE
44	-0.055168	2436.292	546	135	148	29	0	64	0	0
45	-0.056172	2461.18	546	135	147	27	0	67	0	0
46	-0.057601	2495.452	544	135	145	24	0	74	0	0
47	-0.059481	2539.379	542	134	140	29	0	77	0	0
48	-0.060503	2562.457	540	136	134	29	0	83	0	0
49	-0.061935	2594.669	535	136	136	29	0	86	0	0

Table 5.14: Computed limit states for the studied building for - Y direction

The elements which seem to be more stressed and entered further into the plastic zone, as it was expected, are the beams parallel to direction X. It is also observed that the columns (with dimensions 20x20 cm), which are located at the direction Y=0, have entered in the plastic zone. As a results of all the above these columns need to be repaired. The next figure (Figure 5.38) illustrates the diagram between the moments and the plastic rotation for the column Ka.

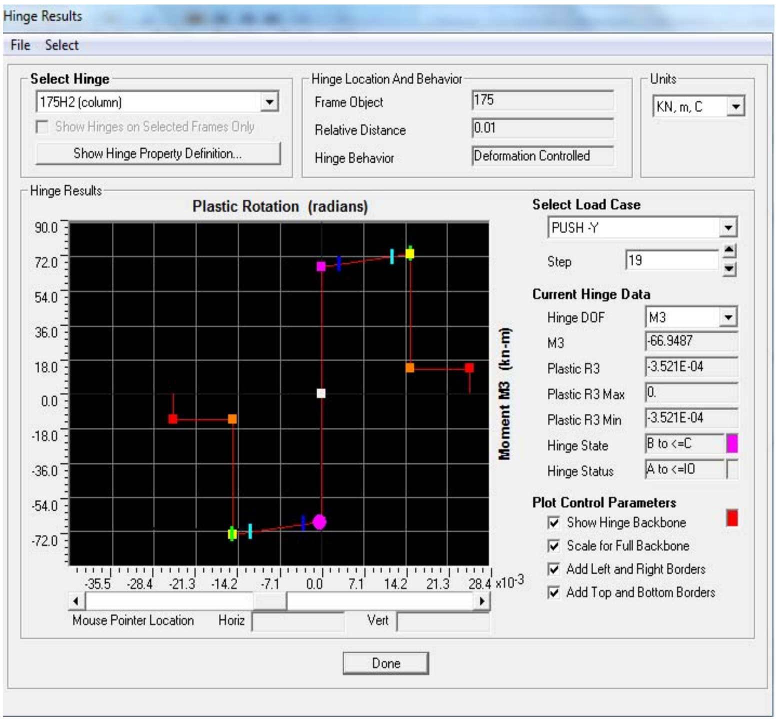

Figure 5.38: Moment - rotation curve

The table 5.15 presents the value of the forces and moment for the column Ka at the performance point.

Column Ka (20x20)			
Step	**P**	**M2**	**M3**
17	-256.971	-13.8734	-46.386
18	-264.657	-14.5933	-49.4811
PERFORMACE POINT	-272.726	-15.2725	-52.1019
20	-280.894	-15.9505	-54.6395
21	-288.763	-16.6733	-57.0465

Table 5.15: The forces and moment for the column Ka at performance point

In the following figure (Figure 5.39) is shown the plan views of the floor 2, 3, 4 & 5 at the performance point at Y direction. Furthermore it can be seen the beams that are embedment in the plate and they have exceeded the limit of the <<life safety >> (yellow color). These beams are K1.1, K1.2, K1.3, K1.4, K1.5, K1.6, K1.7 and K1.8.

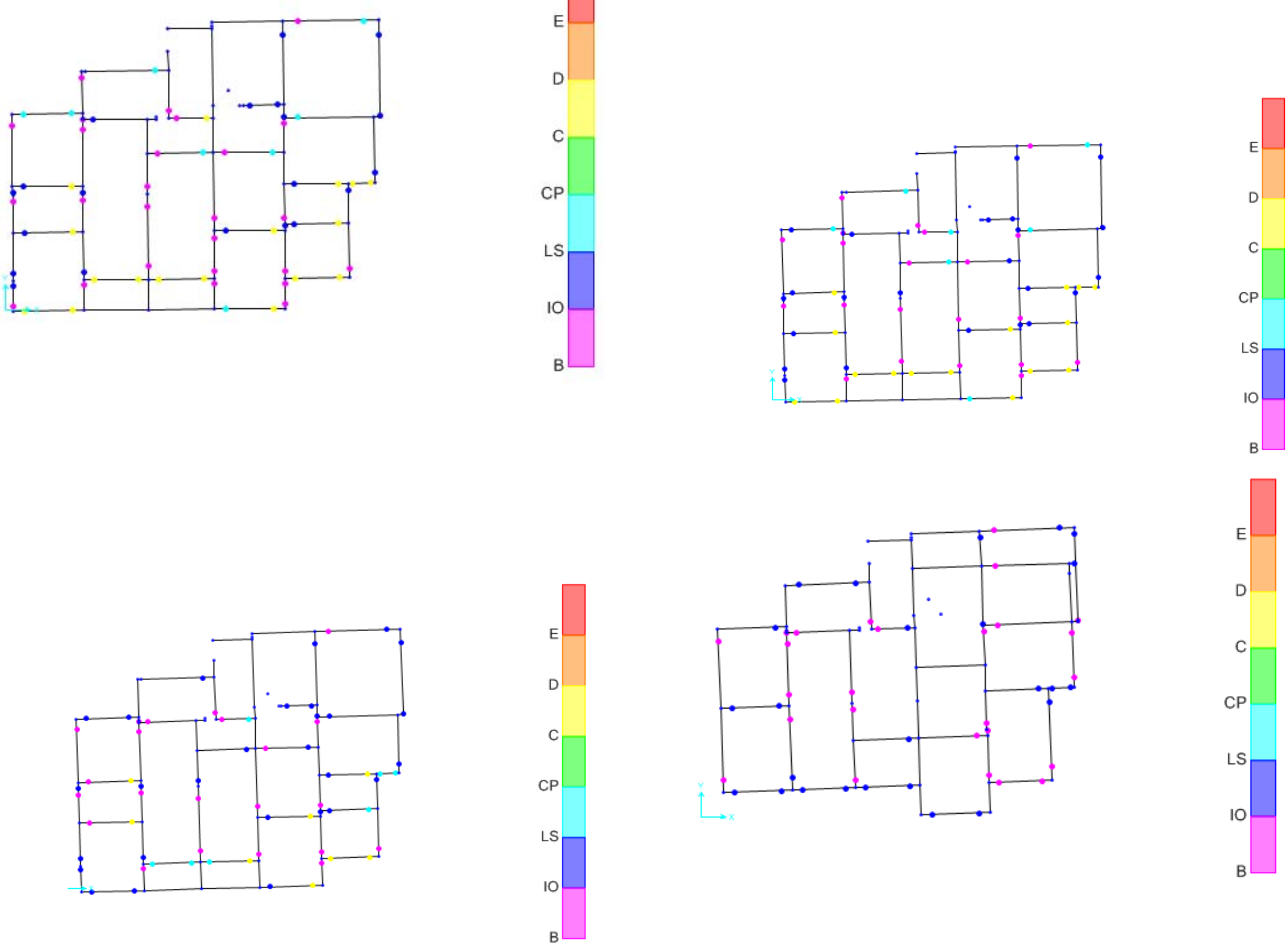

Figure 5.39: The plan views of the floor 2, 3, 4, 5 at the performance point at y direction

It is noted that, at all the floors, more than 20% of the beams are collapsed. The rest of the elements have slight damage, moderate damage, or heavy damage levels. With the condition of brittle elements to be retrofitted, the buildings at this state are assumed to be at Collapse Prevention Performance Level. Functionality of a building at this performance level has risks for its life safety and it should be strengthened. Cost-effective analysis is also recommended for such seismic rehabilitation.

6. Repair and strengthening (retrofitting) of structure

6.1 Introduction

The methods that increase the resistant capacity of structures by various techniques are called retrofitting. Seismic retrofitting is the modification of existing/damaged structures to more resistant to seismic activity, ground motion, or soil failure due to earthquakes buildings. It can also be defined as increasing the seismic resistant of damaged structure by various techniques. The basic concept of retrofitting aims at:

- Up grad of lateral strength of the structure
- Increase the ductility of the building
- Increase of the strength and the ductility

There are two techniques of retrofitting. These are presented below (Figure 6.1):

1. Conventional methods: it is based on increasing the seismic resistance of the existing structure by eliminating or reducing the adverse effect of design or construction.
   - Add new shear walls
   - Add steel bracings
   - Add infill walls
2. Non – Conventional methods based on reduction of seismic demands.
   - Seismic base isolation

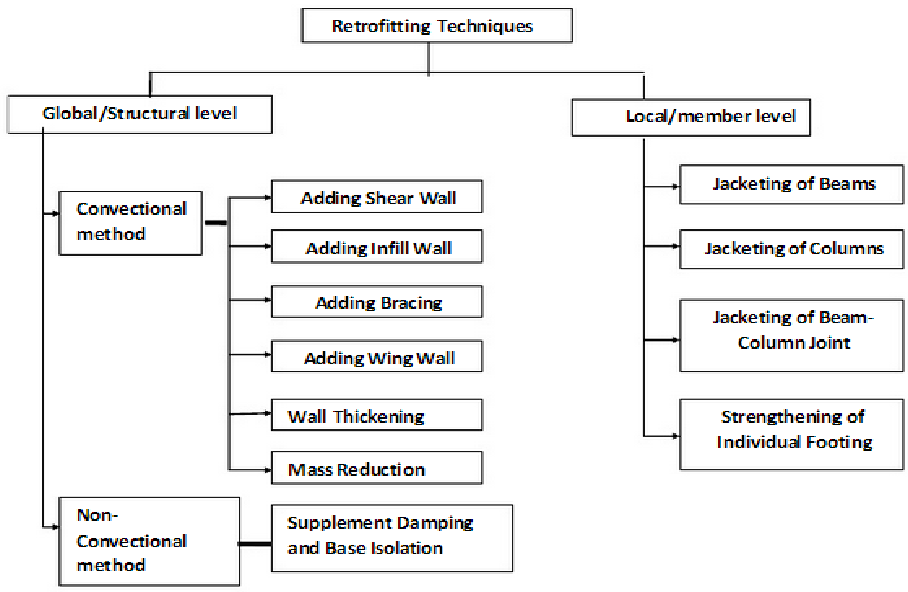

Figure 6.1: Classification of retrofitting techniques

6.2 Reinforced concrete jacket

General: Jacketing is the process whereby a section of an existing structural member is restored to original dimensions or increased in size by encasement using suitable materials. It is the most popularly used method for strengthening of building columns. The most common types of jackets are steel jacket, reinforced concrete jacket, fibre reinforced polymer composite jacket, jacket with high tension materials like carbon fibre and glass fibre.

Concrete Jacketing:

- Involves a thick layer of Reinforced Concrete (RC) in the form of a jacket using longitudinal reinforcement and transverse ties.
- Additional concrete and reinforcement contribute increase the strength.
- Minimum allowable thickness of jacket = 100 mm.
- The sizes of the sections are increased and the free available usable space becomes less.
- Huge dead mass is added.
- The stiffness of the system is highly increased.
- Requires adequate dowelling to the existing column.
- Longitudinal bars need to be anchored to the foundation and should be continuous through the slab.
- Requires drilling of holes in existing column, slab, beams and footings.
- Increase the size, the weight and the stiffness of a column.
- Placement of ties in beam column joints is not practically feasible.
- The speed of implementation is slow.

6.2.1 Reinforced concrete jacketing of column

The reinforced concrete jacketing strengthening method, unlike other techniques, leads to a uniformly distributed increase in strength and stiffness of columns. The durability of the original column is also improved. Finally, this rehabilitation procedure does not require specialized workmanship. Those entire reasons make reinforced concrete (RC) jacketing an extremely valuable choice in structural rehabilitation. The structural behavior of a building rehabilitated by RC jacketing of the columns, like any other strengthening technique, is highly influenced by details.

Reinforced concrete jacketing can be employed as a repair or strengthening scheme. Damaged regions of the existing members should be repaired prior to their jacketing. There are two main purposes of jacketing the columns:

- Increase in' the shear capacity of columns in order to accomplish a strong column – weak beam design,
- to improve the column's flexural strength by the longitudinal steel of the jacket made continuous through the slab system are anchored with the foundation. It is achieved by passing the new longitudinal reinforcement through holes drilled in the slab and by placing new concrete in the beam column joints.

Rehabilitated sections are designed in this way so that the flexural strength of columns should be greater than that of the beams (Figure 6.2).

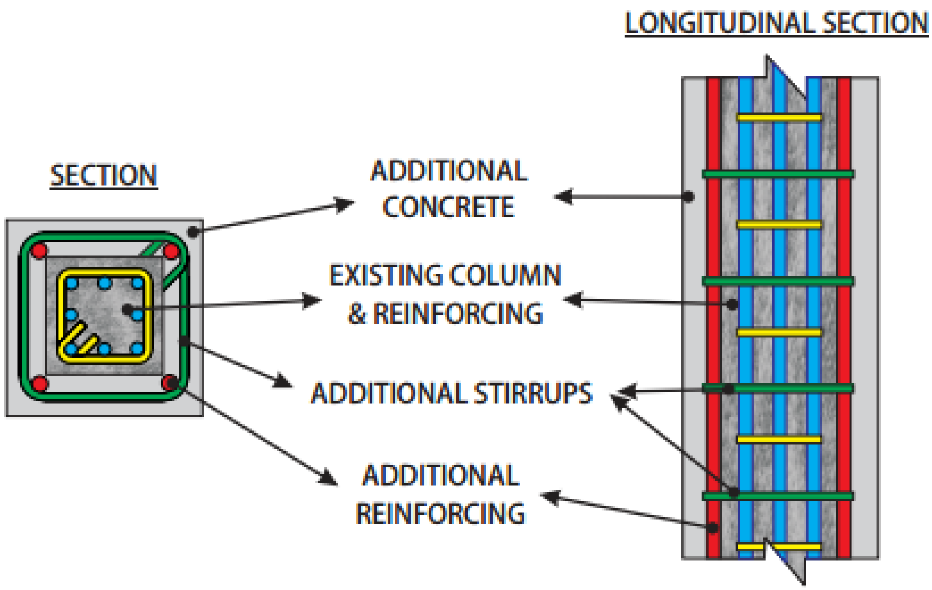

Figure 6.2: Typical reinforced concrete jacketing of column

6.2.1.1Reasons of using jacketing for columns

• Provide adequate temporary supports to all damaged columns and beams down to foundations.

• Cut out damaged concrete.

• Columns should be reinforced with a minimum of 8 bars, with links as recommended. Bars should be lapped at mid height of column with full tension laps (40d min). Bars must be continued and anchored into adjoining members.

• Reinforcement sized to allow for ductile behavior (sized after calculations). Minimum steel should be 16mm diameter bars and links 10mm diameter bars.

• Link spacing to be as specified by design (note, must be close spacing at ends and at lap positions).

• Full continuity should exist for reversal of forces.

• Bar spacing to be restricted to 200mm maximum. All bars to be tied with links.

• Jacketing thickness should be 100mm minimum. Aggregate size should be restricted to 10mm.

6.2.2 Reinforced concrete jacketing of beam

Jacketing of beams is recommended for several purposes as it gives continuity to the columns and increases the strength and stiffness of the construction. While jacketing a beam, its flexural resistance must be carefully computed to avoid the creation of a strong beam – weak column system. In the retrofitted structure, there is a strong possibility of change of mode of failure and redistribution of forces as a result of jacketing of column, which may consequently causes beam hinging. The location of the beam critical section and the participation of the existing reinforcement should be taken into consideration. Jacketing of beam may be carried out under different ways; the most common are one – sided jackets or 3 – and 4 – sided jackets. At several occasions, the slab has been perforated to allow the ties to go through and to enable the casting of concrete. The beam should be jacketed through its whole length. The reinforcement has also been added to increase beam flexural capacity moderately and to produce high joint shear stresses. Top bars crossing the orthogonal beams are put through holes and the bottom bars have been placed under the soffit of the existing beams, at each side of the existing column.

If the analysis shows that the beam reinforcement is not adequate additional reinforcement must be placed and the dimension of the beam can be increase. The typical reinforced concrete jacketing of beam can be seen in Figure 6.3.

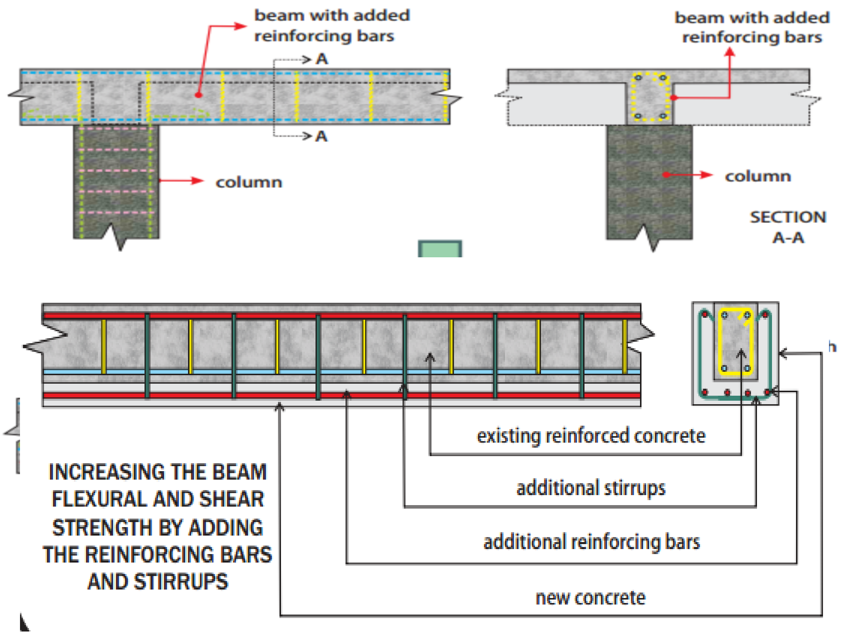

Figure 6.3: Typical reinforced concrete jacketing of beam

6.2.2.1Reasons of using jacketing for beams

• Beams should be reinforced with minimum of 4 bars, with links as recommended. Bars must be continued and anchored into adjoining members.

• Reinforcement sized to allow for ductile behavior (sized after calculations). Minimum steel should be 16mm diameter bars and links 10mm diameter bars.

• Link spacing to be as recommended (note, close spacing at ends and at lap positions).

• Full continuity should exist for reversal of forces.

• All main bars to be tied with links.

• Jacketing thickness should be 100mm minimum. Aggregate size should be restricted to 10mm.

6.3 Structures with increased reinforcement

Initially, before retrofitting the construction, several beams and columns were designed with more reinforcement. The change in the behavior of the structure is as follows. The selected columns and beams are defined in the following tables and figures. From the previous pushover analysis these beams and columns were identified as not adequate.

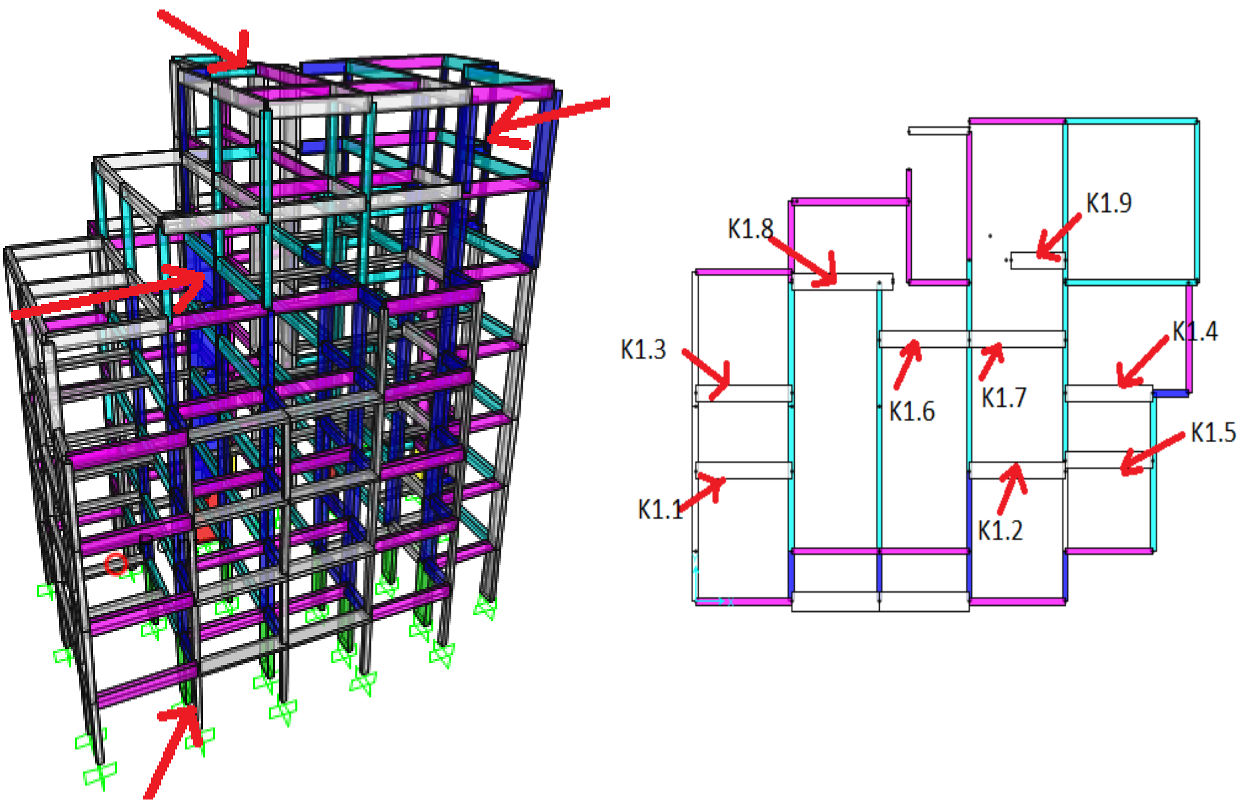

Figure 6.4: The columns and beams with additional reinforcement

The following table lists the columns in which the reinforcement has changed (increased). These changes were performed up to the fourth floor.

Column name	Existing reinforcement	New reinforcement
Ka (20x20)	4Y14	4Y20
K10 (20x120)	10Y16	16Y16
K13 (100x20)	10Y14	16Y16
K21 (20x70)	10Y16	10Y20

Table 6.1: The existing and the new reinforcement for column

The table below illustrates the beams the reinforcement of which has changed (increased).

K1.1,K1.2 ,K1.3,K1.4,K1.5,K1.6,K1.7,K1.8,K1.9		Existing reinforcement	New reinforcement
ground floor	top	5Y12	6Y14
	bottom	5Y12	6Y14
first floor	top	5Y12	6Y14
	bottom	5Y12	6Y14
second floor	top	5Y12	6Y14
	bottom	5Y12	6Y14
third floor	top	5Y12	6Y14
	bottom	5Y12	6Y14
thourth floor	top	5Y12	6Y14
	bottom	5Y12	6Y14
fithf floor	top	5Y12	6Y14
	bottom	5Y12	6Y14
top floor	top	5Y12	6Y14
	bottom	5Y12	6Y14

Table 6.2: The existing and the new reinforcement of beams

6.3.1 Analysis of the results

Direction +X: This analysis was completed in 96 steps and the performance point was set between steps 58 and 59. The performance point Sd is 0.149 m. The resulting pushover curve, in terms of Base Shear – Roof Displacement (V-Δ), is given in figure below for + X direction**.**

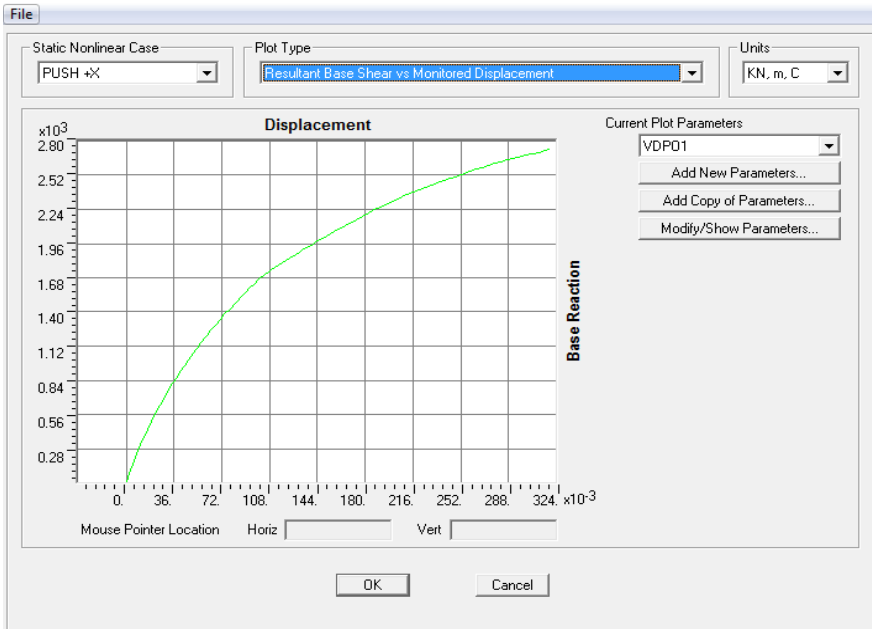

*Figure 6.5***:** *The new pushover curve, in terms of Base Shear – Roof Displacement for direction +X*

Comparison of the pushover curves in the +X (initial structure) and + X (new structure) directions shows that the results from these two curves are similar. The max point for +X (initial structure) direction is V =2959 kN and Δ = 0.34 m and for +X direction (new structure) is V = 2932 kN and Δ = 0.319 m respevtively. Comparing the initial with the new structure can be concluded that the new structure receives about the same amount of forces but the displacement of it is less by approximately 3 cm.

The performance point is the point where the capacity curve crosses the demand curves according to ATC-40. The next figure demonstrates the performance point for push over curve at +X direction for the latest building.

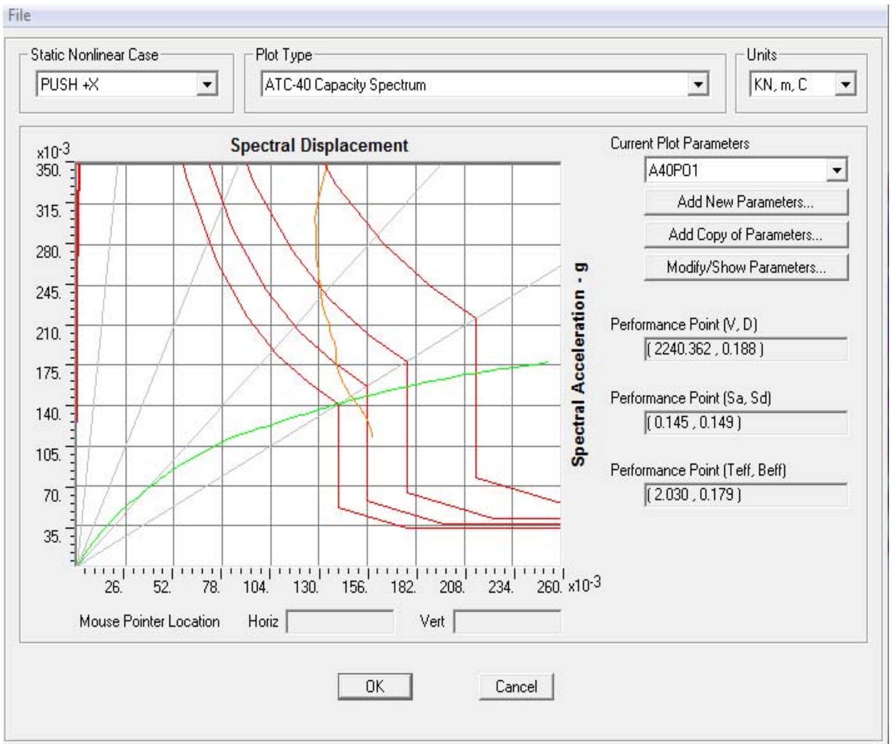

Figure 6.6:Pushover curve for +X direction

The table below displays some of the steps of the analysis. For each step can be seen the details for the capacity and demand curve for the equivalent one degree of freedom system. Furthermore in that table there is a comparison between the results of the first construction and the strengthened one.

Capacity curve			Capacity and demand curve					
step	**diplacement (m)**	**Base force KN**	**Teff**	**Beff**	**SdCapacity(m)**	**SaCapacity**	**SdDemand(m**	**SaDemand**
INITIAL STRUCTURE								
59	0.174205	2443.933	1.99574	0.16539	0.147629	0.149211	0.150341	0.151952
60	0.191153	2332.06	2.00671	0.16637	0.150826	0.150781	0.150828	0.150783
NEW STRUCTURE								
58	0.185463	2426.32	2.02256	0.17847	0.146565	0.144234	0.148268	0.14591
59	0.189269	2446.67	2.03395	0.17946	0.149581	0.145558	0.148815	0.144813

Table 6.3: Computation of the new performance point for +X direction

Comparing the results above it is observed that the performance point has been reduced slightly by about half a centimeter while the base shear in the new structure is almost 5% greater to the initial structure.

In the following table the number of element which entered in the plastic zone are displayed. Moreover there is a comparison between the results of the initial drawing and the reinforced one.

Step	Displacement (m)	BaseForce KN	AtoB	BtoIO	IOtoLS	LStoCP	CPtoC	CtoD	DtoE	BeyondE
INITIAL STRUCTURE										
59	0.187104	2307.689	631	109	99	67	0	16	0	0
60	0.191153	2332.06	631	108	91	65	0	27	0	0
NEW STRUCTURE										
58	0.185463	2426.32	612	127	116	51	0	16	0	0
59	0.189269	2446.67	611	125	111	53	0	22	0	0

Table 6.4: Computed limit states for the strengthened building for + X direction

From the above table it is observed that the numbers of elements which entered in the plastic zone have slightly reduced. In the new building the number of elements with yellow color is reduced by 18% as compared with the original structure. All of these results to an improvement of the behavior of the structure.

Direction +Y: This analysis was completed in 52 steps and the performance point was set between steps 45 and 46 of it. The performance point Sd is equal with 0.154m. The resulting pushover curve, in terms of Base Shear – Roof Displacement (V-Δ) for +Y direction, is given in figure below (Figure 6.7).

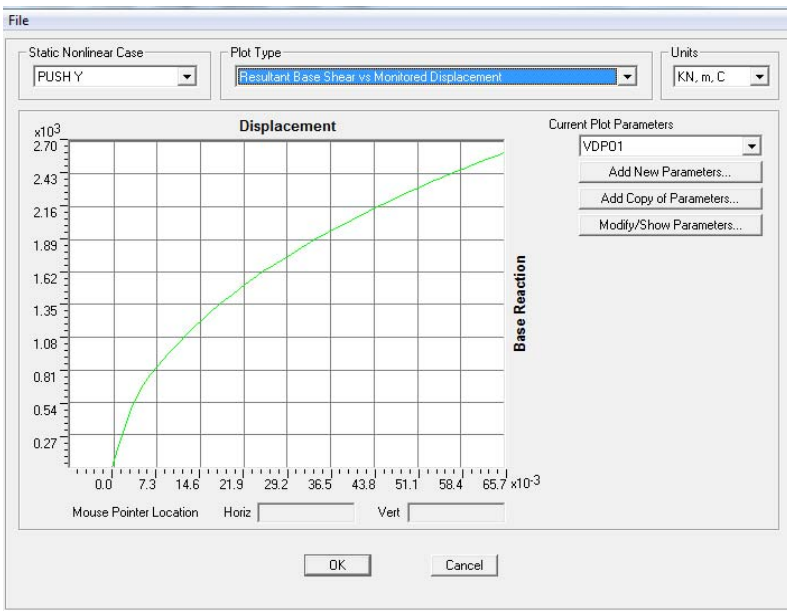

Figure 6.7: The new pushover curve, in terms of Base Shear – Roof Displacement for direction +Y

According to ATC – 40 the performance point is the point where the capacity curve crosses the demand curves. In the figure6.8 the performance point for push over curve at +Y direction for the structure with the increased reinforcement is presented.

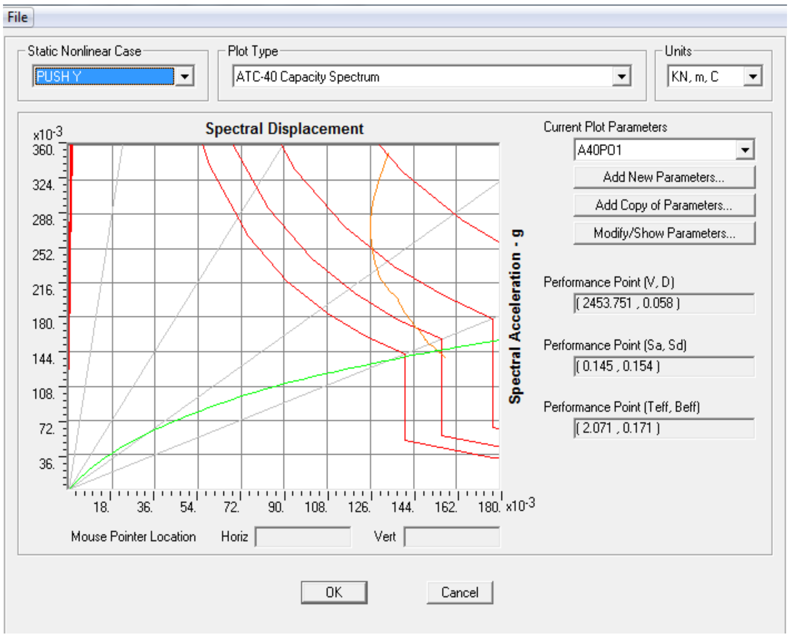

Figure 6.8: New capacity curve for +Y direction

In the table beneath some of the steps from the pushover analysis can be observed. For each step the details for the capacity and demand curve for the equivalent one degree of freedom system are displayed. In addition in the table a comparison between the results of the two buildings is presented.

Direction +Y								
Capacity curve			**Capacity and demand curve**					
step	**diplacement (m)**	**Base force KN**	**Teff**	**Beff**	**SdCapacity(m)**	**SaCapacity**	**SdDemand(m)**	**SaDemand**
INITIAL STRUCTURE								
43	0.055212	2543.269	2.02273	0.15599	0.15287	0.150413	0.155535	0.153035
44	0.056444	2571.637	2.03575	0.15746	0.156766	0.15228	0.156038	0.151572
NEW STRUCTURE								
45	0.057608	2451.135	2.06928	0.1707	0.153815	0.144611	0.15413	0.144907
46	0.059242	2483.992	2.08545	0.17189	0.158541	0.146751	0.154905	0.143386

Table 6.5: Computation of the new performance point for +Y direction

Comparing the above results it can be concluded that the performance point is reduced slightly by about half centimeter while the base shear in the new structure is smaller about 3% compared with the original structure.

After that the table with the elements that has entered the plastic zone is shown. Furthermore in that table there is a comparison between the results of the two structures.

Step	Displacement (m)	BaseForce KN	AtoB	BtoIO	IOtoLS	LStoCP	CPtoC	CtoD	DtoE	BeyondE
INITIAL STRUCTURE										
43	0.055212	2543.269	582	106	134	45	0	55	0	0
44	0.056444	2571.637	579	105	130	48	0	60	0	0
NEW STRUCTURE										
45	0.057608	2451.135	541	128	176	32	0	45	0	0
46	0.059242	2483.992	537	127	171	40	0	47	0	0

Table 6.6: Computed limit states for the new studied building for + Y direction

From the above table it is observed that the number of elements which entered the plastic region have been slightly reduced. In the new structure the number of elements with yellow color is reduced by 22% as compared to the original structure. As a result, in this direction, there is an improvement of the behavior of the entire building.

6.4Analysis of structures retrofitted with jacketing of beams and columns

Objective: The main scope of this thesis is to assess and improve the earthquake resistance of the building. The retrofitting system aims at increasing of the strength and the stiffness of the structure. The basic deficiencies marked on the existing building are:

1. Inability to sustain static and earthquake loads in certain columns and beams.
2. Significant torsion under the dead loads.

The fundamental periods of the structure are:

- T1= Tx = 1.45 sec
- T2 =0.98 sec
- T3 = Ty=0.81 sec

From all the above the main effort was to reduce the torsion, which is a major deficiency of the structure as it may increase damage to the concrete members.

Introduction:

This part describes the process and the effect of using concrete jacketing to some columns (it can be seen in Figure 6.9) and in all beams. Concrete jacketing with thickness 10 cm at the beams and columns is considered. As mentioned above jacketing a beam, increases its flexural resistance and thus must be carefully computed to avoid the formation of a strong beam – weak column system. The reinforced concrete jacketing strengthening method, unlike other techniques, leads to a uniformly distributed increase in strength and stiffness of columns. In the figure below the beams and columns which are retrofitted are highlighted. Modeling of the structure in the program SAP 2000, which was used for the analysis, is also presented.

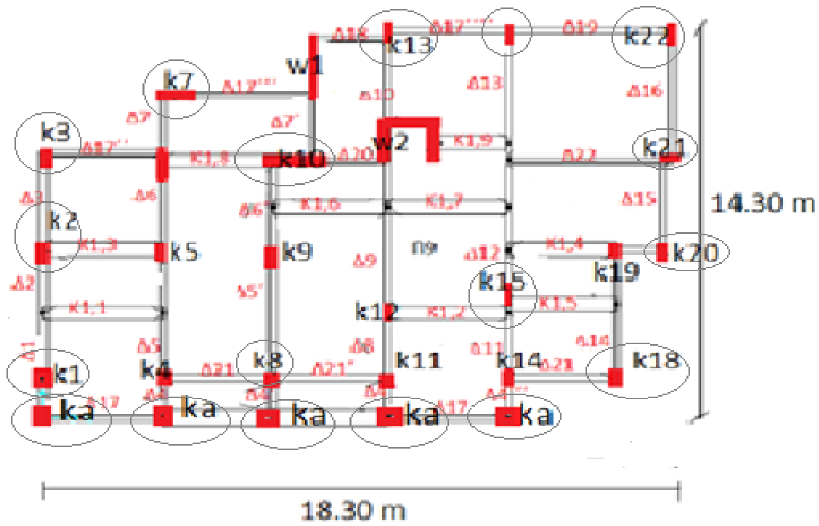

Figure 6.9: The columns which were repaired

As it is observed above are 18 out of the 27 columns are reinforced. All the external and some of the internal columns were strengthen and this was performed up to the 4^{th} floor. In all the downstream beams a concrete jacket of 10cm thickness was considered. With the use of the program a new section was formed constituted from the initial and the new material. The new materials are concrete C20/25 and steel S500. Their properties are shown underneath.

CONCRETE C20/25		STEEL S500	
Charactiristics		Charactiristics	
Fck (MPa)	20	E (GPa)	200
E (GPa)	29	Poisson's ratio (V)	0.3
Poisson's ratio (V)	0.2	minimun yield stress Fy (KN/m²)	500000
Fctm (MPa)	2.2	minimun tensile stress (Fu KN/m²)	600000
Specific weight (KN/m³)	25	Specific weight (KN/m³)	78.5

Table 6.7: The new materials of the structure

The initial and the final dimensions of the columns are presented below:

Floor	Ka		K1.2.3.8.15.17.18.20.22		K7	
	initial sectio	new section	ini.section	new section	ini.section	new section
gound floor	20x 20 4 Y14	40x40 4Y14 &4Y20	70x20 10Y16	90x40 10Y16 & 13Y20	20X60 10Y16	40X80 10Y16 &12Y20
floor 1	20x 20 4 Y14	40x40 4Y14 &4Y20	70x20 10Y16	90x40 10Y16 & 13Y20	60X20 10Y16	80X40 10Y16 &12Y20
floor 2	20x 20 4 Y14	40x40 4Y14 &4Y20	70x20 10Y16	90x40 10Y16 & 13Y20	60X20 10Y16	80X40 10Y16 &12Y20
floor 3	20x 20 4 Y14	40x40 4Y14 &4Y20	70x20 10Y16	90x40 10Y16 & 13Y20	60X20 10Y16	80X40 10Y16 &12Y20
floor 4	20x 20 4 Y14	40x40 4Y14 &4Y20	70x20 10Y16	90x40 10Y16 & 13Y20	60X20 10Y16	80X40 10Y16 &12Y20

Floor	K10		K13		K16		K21	
	ini.section	new section	ini.section	new section	ini.section	new section	ini.section	new section
gound floor	20X120 10Y16	40X140 10Y14&14Y20	100X20 8Y14	120X40 8Y14& 14Y20	20X80 10Y16	40X100 10Y16&14Y20	80X20 10Y16	100x40 10Y16&14Y20
floor 1	20X120 10Y16	40X140 10Y14&14Y20	100X20 8Y14	120X40 8Y14& 14Y20	20X80 10Y16	40X100 10Y16&14Y20	80X20 10Y16	100x40 10Y16&14Y20
floor 2	20X120 10Y16	40X140 10Y14&14Y20	100X20 8Y14	120X40 8Y14& 14Y20	20X80 10Y16	40X100 10Y16&14Y20	80X20 10Y16	100x40 10Y16&14Y20
floor 3	20X120 10Y16	40X140 10Y14&14Y20	100X20 8Y14	120X40 8Y14& 14Y20	20X80 10Y16	40X100 10Y16&14Y20	80X20 10Y16	100x40 10Y16&14Y20
floor 4	20X120 10Y16	40X140 10Y14&14Y20	100X20 8Y14	120X40 8Y14& 14Y20	20X80 10Y16	40X100 10Y16&14Y20	80X20 10Y16	100x40 10Y16&14Y20

Table 6.8: Initial and new sections for the columns

6.4.1 The procedure in Sap2000:

This part of the study describes the introduction of new materials and the design of the new, modified sections in the program (Figure 6.10):

- Firstly the designer inserts the new material in the program:

Define → materials → Add new material

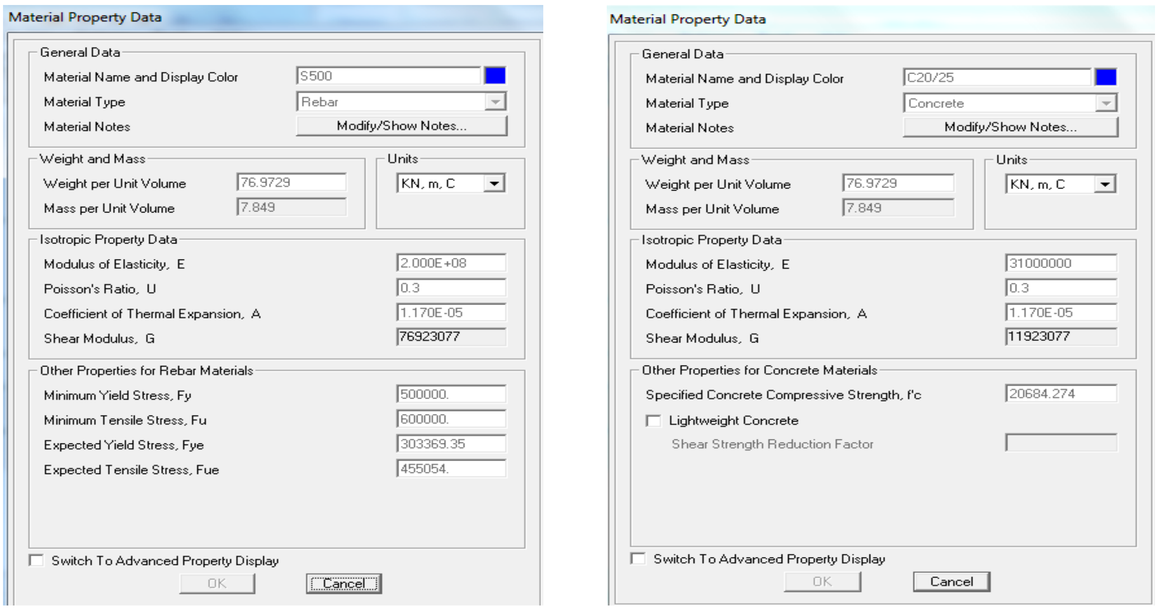

Figure 6.10: Introduction of the new materials in the program

- After that with the use of the section designer (which is useful for the modeling, analysis, and design of reinforced-concrete columns and beams) the new sections are introduced. As mentioned before around the perimeter of the column reinforced concrete jacketing with thickness 10 cm was placed. Below the modified columns K7 and K13are presented (Figures 6.11 & 6.12):

.

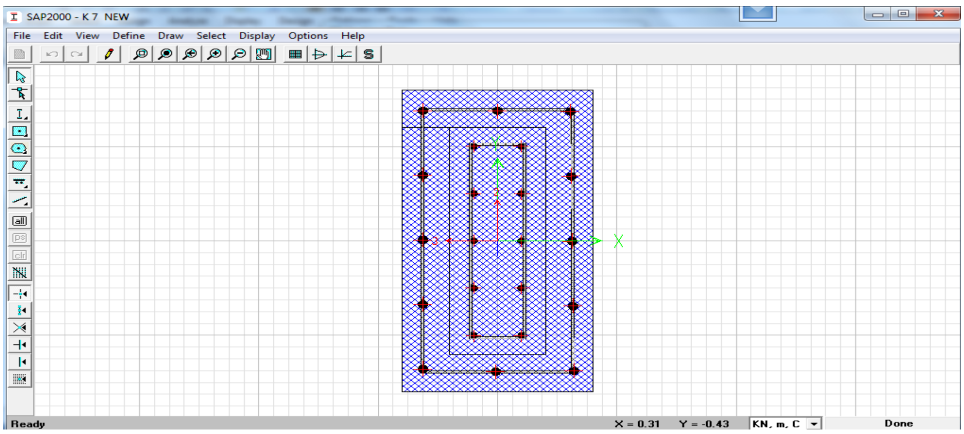

Figure 6.11: Retrofitting of Column K7 using section designer

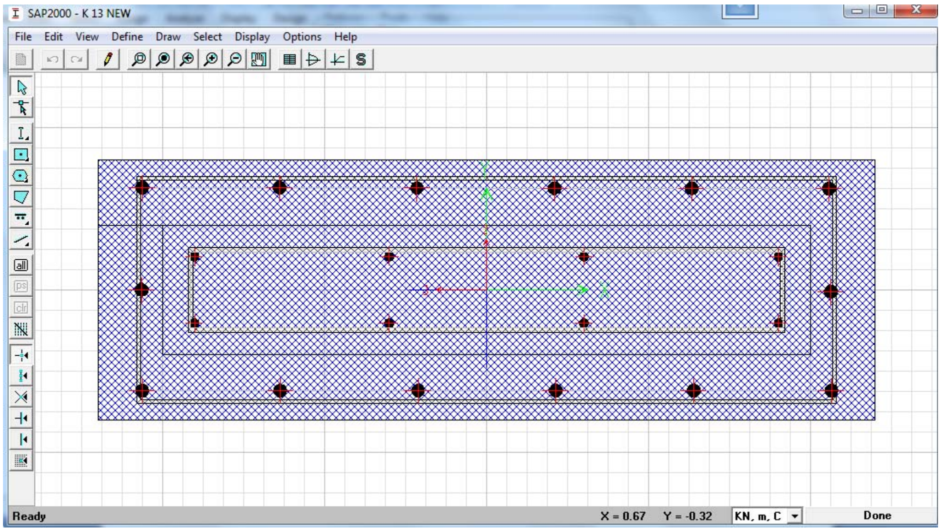

Figure 6.12: Retrofitting of Column K13 using section designer

- Moreover concrete jacketing with 10cm thickness is placed at the bottom of the beams. Modified beams can be observed in Figure 6.13:

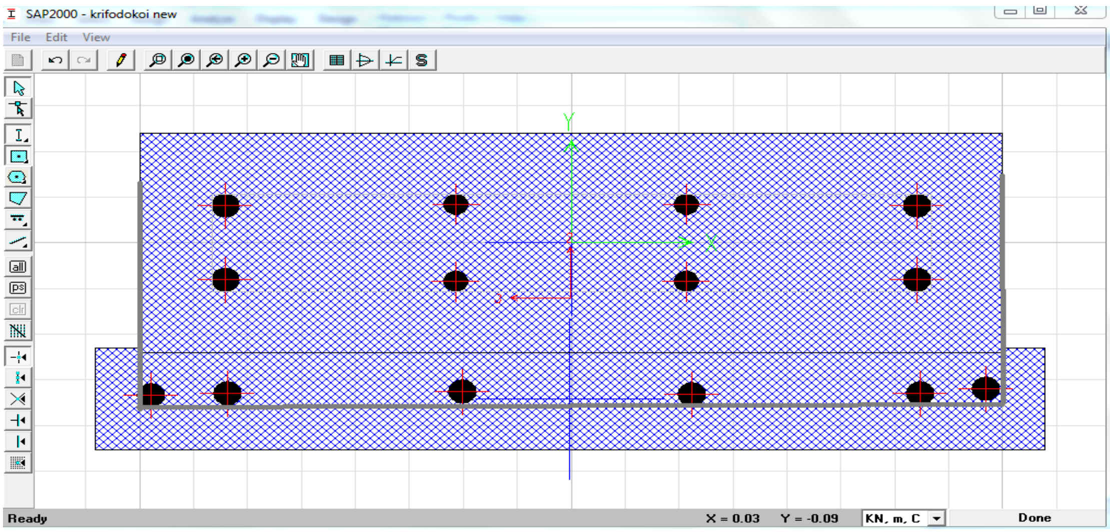

Figure 6.13: Retrofitting of beam using section designer by adding a 10 cm layer of shotcrete with reinforcement and stirrups

6.4.2 Results

In this part, the behavior of the existing and the strengthened structure, after the retrofitting, is being compared. First the maximum displacements of the two buildings and then the pushover results are compared.

- Initially the maximum displacements after the retrofitting at the joint 452 are compared. The deformations are related to the load combination G+0.3Q. The results are shown beneath **:**

Joint	U1(m)	U2(m)	U3(m)	R1(Radians)	R2(Radians)	R3(Radians)
INITIAL STRUCTURE						
452	0.001124	-0.006449	-0.002321	-0.000026	-0.000777	-0.000835
NEW STRUCTURE						
452	0.0009155	-0.001656	-0.00104	0.000034	-0.000567	-0.000326

Table 6.9: The maximum displacements before and after retrofitting of the structure

- The following table displays the values of the fundamental period of vibration of the existing and the new structure, after the retrofitting.

	T1 (Tx)	T2	T3 (Ty)
INITIAL STRUCTURE	1.452.864	0.982434	0.814143
NEW STRUCTURE	1.089188	0.74527	0.576446

Table 6.10: The fundamental periods before and after retrofitting of the structure

From the above it concluded that the new structure has better behavior characteristics and therefore better behavior.

<u>Direction +X:</u> This analysis was completed in 96 steps and the performance point was set between steps 41 and 42. The performance point Sd is equal to 0.102 m. The resulting pushover curves, in terms of Base Shear – Roof Displacement (V-Δ) for the existing and the new structure in + X direction, are given in the next figure (Figure 6.14):

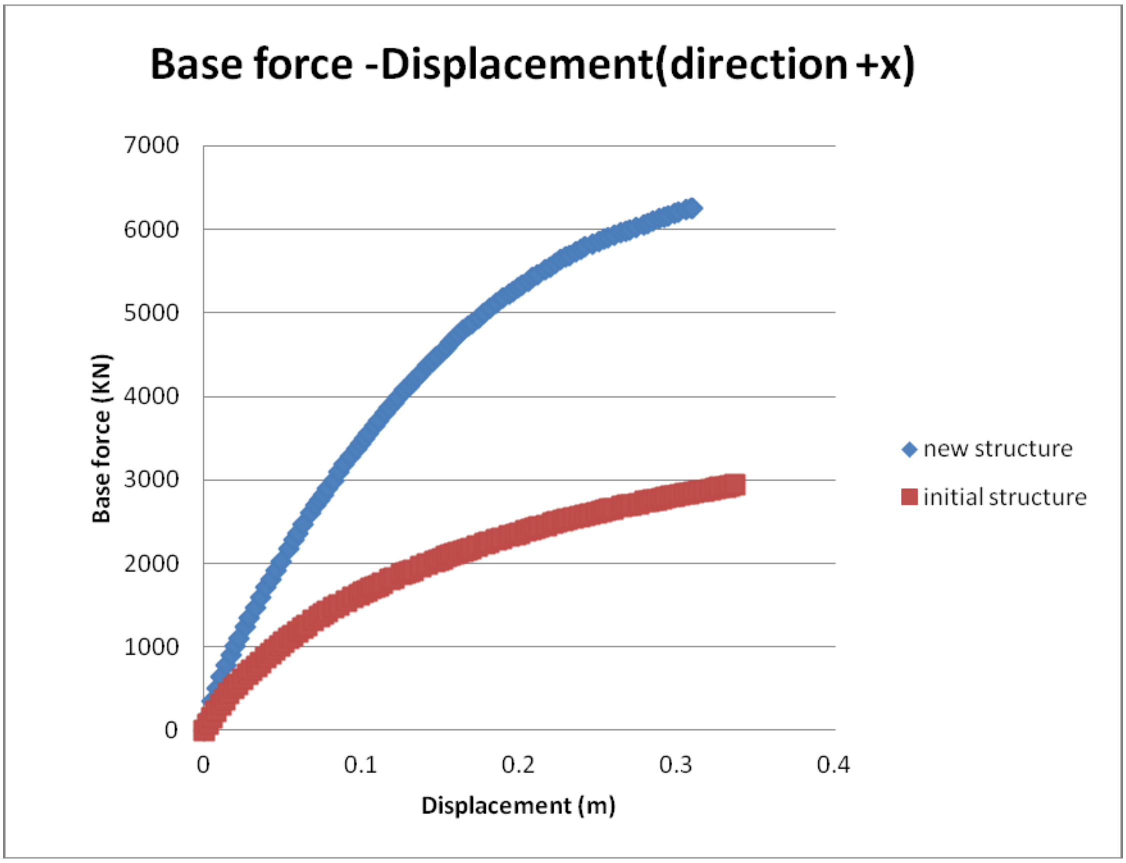

Figure 6.14: The comparison of the pushover in terms of Base Shear – Roof Displacement for direction +X before and after retrofitting

The comparison of the pushover curves in the +X (initial structure) and + X (new structure) directions shows that the results from these two curves are substantially different with the new structure's capacity increased by more than two times the initial value. The max point for initial structure is V =2959 kN and Δ = 0.34 m and for new structure is V = 6256 kN and Δ = 0.319 m. From those can be concluded that the new structure can sustain more forces than the initial structure. As a consequence the behavior of the building is improved.

The performance point is the point where the capacity curve crosses the demand curves according to ATC – 40. The following figure illustrates the performance point for pushover curves for the initial and new structure at +X direction.

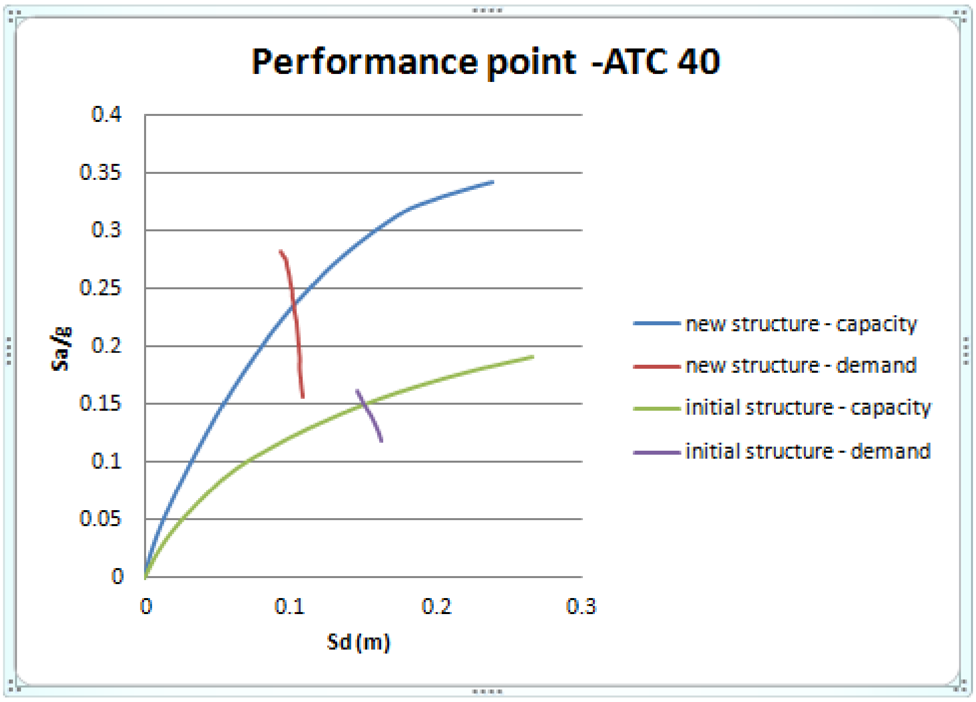

Figure 6.15 The comparison of the capacity curve for +X direction before and after retrofitting

Table 6.11 shows some of the steps from the pushover analysis. From each step the details for the capacity and demand curve for the equivalent one degree of freedom system is presented. Furthermore there is a comparison between the results of the initial and the new structure.

Direction +X								
Capacity curve			Capacity and demand curve					
step	diplacement (m)	Base force KN	Teff	Beff	SdCapacity(m)	SaCapacity	SdDemand(m	SaDemand
INITIAL STRUCTURE								
59	0.174205	2443.933	1.995744	0.165385	0.147629	0.149211	0.150341	0.151952
60	0.191153	2332.06	2.006709	0.166366	0.150826	0.150781	0.150828	0.150783
NEW STRUCTURE								
41	0.129832	4129.264	1.317386	0.117292	0.100418	0.232929	0.10227	0.237226
42	0.132342	4181.577	1.321905	0.118088	0.102358	0.235809	0.102447	0.236015

Table 6.11: Computation of the new performance point for +X direction

Comparing the new point of performance V, D (4181. 0,132) with point of performance of the corresponding analysis of the original structure, V, D (2332, 0,191) it is observed that there is a significant reduction in target displacement by 4 – 5cm. Moreover there is a significant increase in the base shear by 460kN. This means that in the +X direction the stiffness of the structure has increased considerably due to the retrofitted sections. This is confirmed by the Figure 6.16 showing that the comparative curves of the original and the retrofitted building with the second curve (new structure) have approximately doubled the stiffness of the first curve.

In the table below the whole structure the state of the elements at the performance point before and after the retrofitting of the sections is presented.

Step	Displacement (m)	BaseForce KN	AtoB	BtoIO	IOtoLS	LStoCP	CPtoC	CtoD	DtoE	BeyondE
INITIAL STRUCTURE										
59	0.187104	2307.689	631	109	99	67	0	16	0	0
60	0.191153	2332.06	631	108	91	65	0	27	0	0
NEW STRUCTURE										
41	0.129832	4129.264	657	100	164	1	0	0	0	0
42	0.132342	4181.577	655	99	165	3	0	0	0	0

Table 6.12: Computed limit states for the new studied building for + X direction

From the previous table it is observed that the number of elements that entered in the plastic zone has been reduced appreciably. At the new design there are no elements that exceed the limit of level <<LIFE SAFETY >>. In the new building the numbers of elements with blue color are very few as compared to the original structure. Furthermore there are no elements with yellow color. Therefore in the direction +X the retrofitting of the elements is considered successful and now the structure satisfies the intended objective.

Direction +Y: This analysis was completed in 68 steps and the performance point was set between steps 28 and 29 of it. The performance point Sd is equal with 0.103m. The resulting pushover curves, in terms of Base Shear – Roof Displacement (V-Δ) for +Y direction before and after retrofitting, are given in figure below**:**

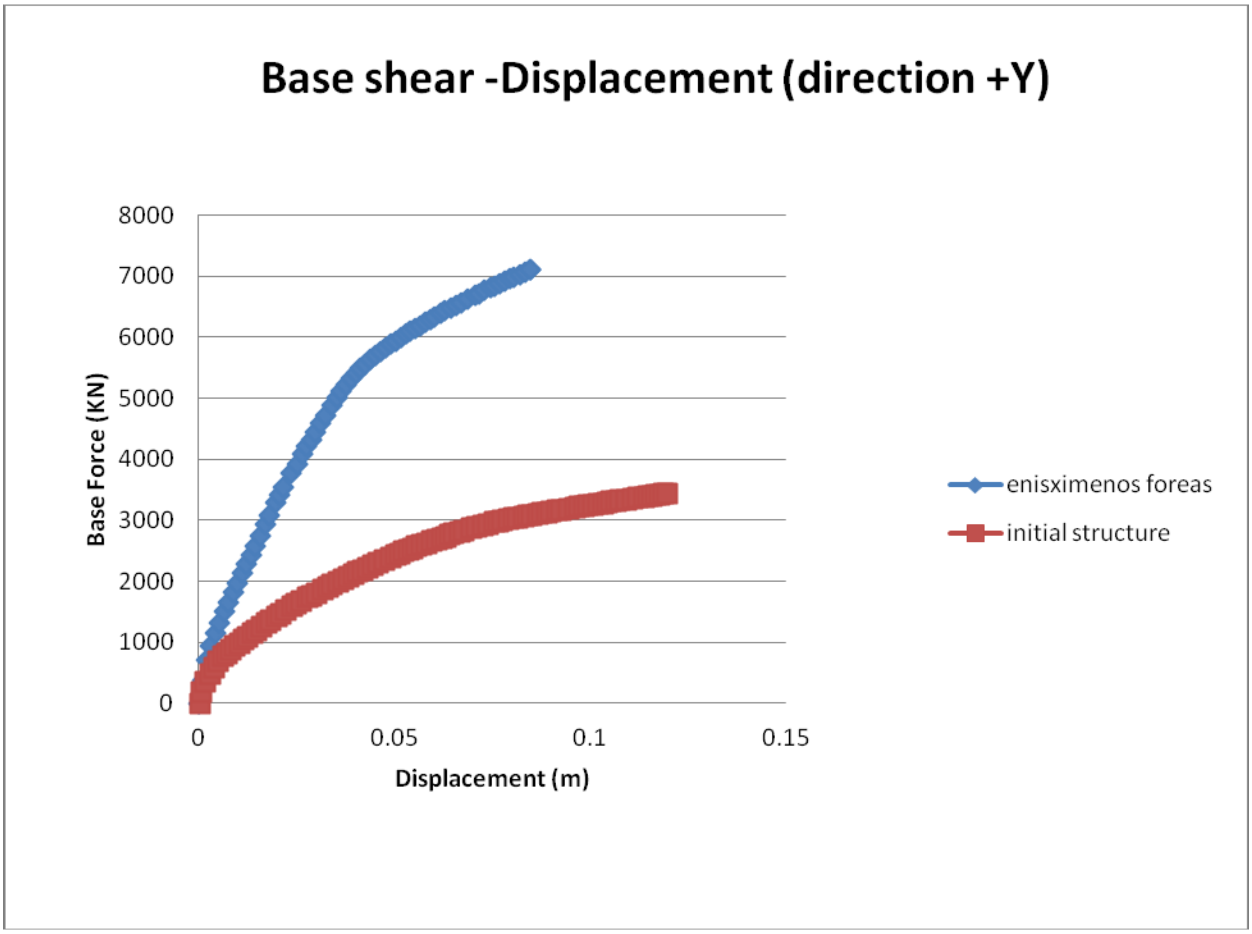

Figure 6.16: The comparison of the pushover in terms of Base Shear – Roof Displacement for direction +Y before and after retrofitting

Figure 6.16 illustrates that the pushover curve for the retrofitted building that has doubled the stiffness of the capacity curve as compared to the initial structure. The max point for initial structure is V = 3437.95 kN and Δ = 0.119 m and after the latest drawing is V = 7115.94 kN and Δ = 0.08490 m. Comparing the two structures it can be concluded that the new structure is significantly more capable to receive additional loads. Also the maximum displacement is greatly reduced as compared to the initial structure. So the behavior of the structure due to retrofitted sections overall is improved.

Table 6.13 displays the base shear force and the displacement at the performance point for the existing and the new structure, after the retrofitting.

Direction +Y								
Capacity curve			**Capacity and demand curve**					
step	**diplacement (m)**	**Base force KN**	**Teff**	**Beff**	**SdCapacity(m)**	**SaCapacity**	**SdDemand(m)**	**SaDemand**
INITIAL STRUCTURE								
43	0.055212	2543.269	2.022733	0.155985	0.15287	0.150413	0.155535	0.153035
44	0.056444	2571.637	2.035753	0.157459	0.156766	0.15228	0.156038	0.151572
NEW STRUCTURE								
28	0.034038	4887.004	1.321208	0.115596	0.099326	0.229065	0.103077	0.237715
29	0.035275	5007.597	1.331244	0.117723	0.103384	0.234843	0.103359	0.234786

Table 6.13: Computation of the new performance point for +Y direction

In the table underneath a substantial reduction of the target displacement at 2.1 cm is observed. Additionally the base shear is doubled. The above means that in this direction, the stiffness of the whole construction has increased considerably. This is confirmed by the following figure, which demonstrates the performance point for push over curve at +Y direction for both buildings.

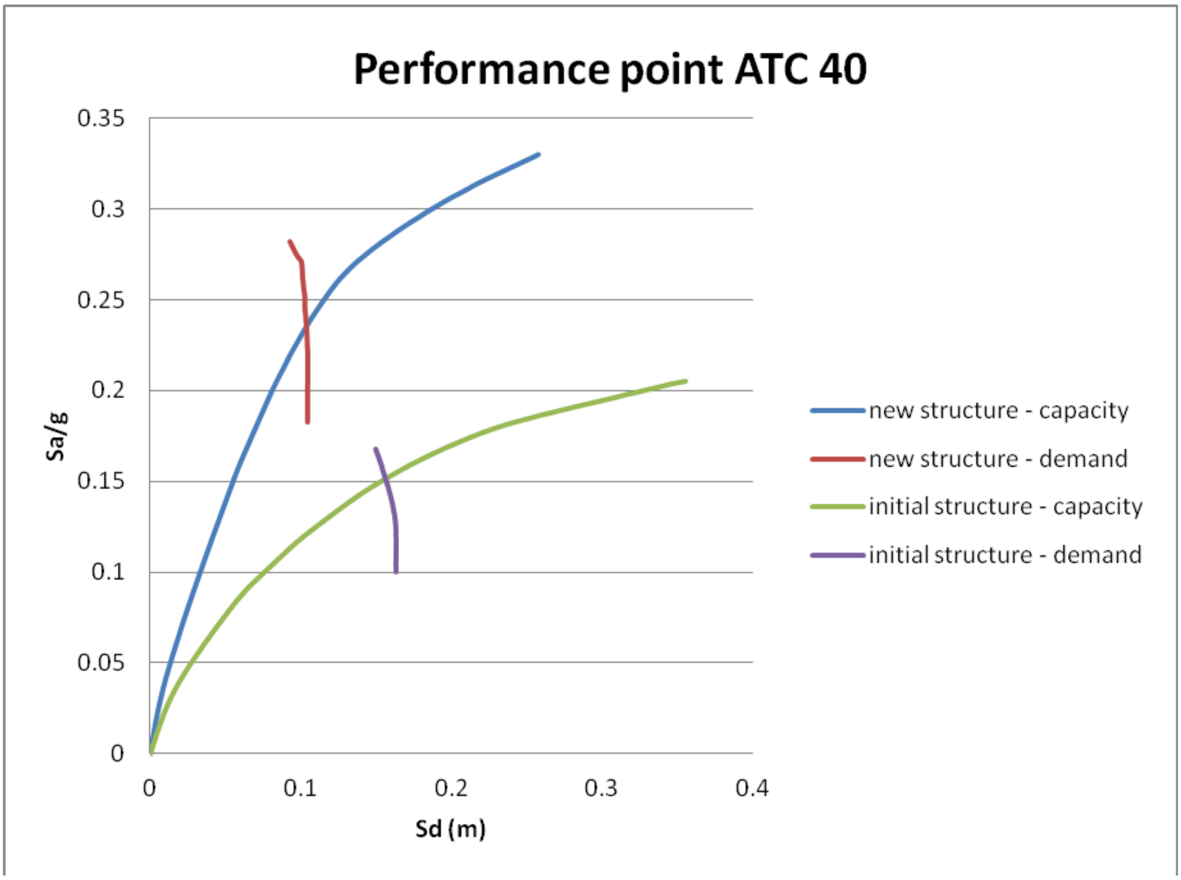

Figure 6.17: The comparison of the capacity curve for +Y direction before and after retrofitting

Moreover table with the number of elements which entered in the plastic zone are presented. In that table the results of the initial structure and the new structure are compared.

Step	Displacement (m)	BaseForce KN	AtoB	BtoIO	IOtoLS	LStoCP	CPtoC	CtoD	DtoE	BeyondE
INITIAL STRUCTURE										
43	0.055212	2543.269	582	106	134	45	0	55	0	0
44	0.056444	2571.637	579	105	130	48	0	60	0	0
NEW STRUCTURE										
28	0.034038	4887.004	589	144	162	27	0	0	0	0
29	0.035275	5007.597	585	144	158	35	0	0	0	0

Table 6.14: Computed limit states for the new studied building for + X direction

From the previous table it is observed that the number of elements that entered the plastic zone have been reduced appreciably. At the new structure there are no elements which are exceed the limit of level <<LIFE SAFETY >>. The figure beneath (Figure 6.18) compares the overall yielding pattern of the initial and new structure at the performance point.

- New structure Initial structure

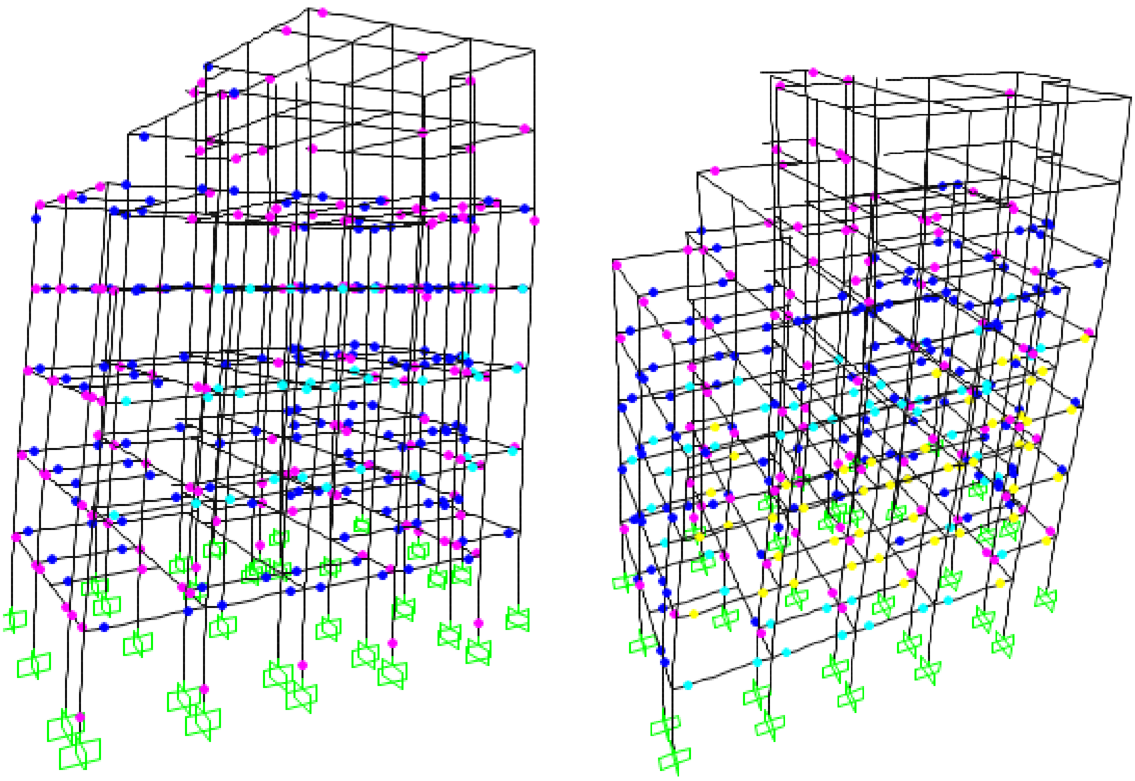

Figure 6.18 :The overall yielding pattern before and after retrofitting

6.5 Non Linear Dynamic Time History Analysis

Dynamic analysis is clearly more complex and time consuming than a static analysis. The structure model before and after retrofitting is subjected to load time history. Their maximum displacements and stresses have been plotted for the two models. The load time history used was the Northridge earthquake.

Non – linear dynamic analysis utilizes the combination of ground motion records with a detailed structural model definition, therefore is capable of producing results with relatively low uncertainty. In non – linear dynamic analysis, the detailed structural model subjected to a ground motion record produces estimates of component deformations for each degree of freedom in the model and the modal responses are combined using schemes such as the square root of sum of squares(SRSS). In this analysis, the non – linear properties of the construction are considered as part of a time domain analysis. This approach is the most rigorous and is required by some building codes for buildings of unusual configuration or of special importance. However, the calculated response can be very sensitive to the characteristics of the individual ground motion used as seismic input.

The non – linear direct – integration time – history analysis was continued from a nonlinear static analysis or another direct – integration time – history nonlinear analysis. It is strongly recommended for the designer to select the same geometric nonlinearity parameters for the both cases.

6.5.1 Earthquake for non linear time history analysis

In the present case study for non linear time history analysis, the earthquake of Northridge at 1994 record P088 it was used by applying the accelerograms in directions X and Y. The accelerograms that used come from measurements made at the surface of the Old Ridge Rout station.

Primarily, the data of the earthquake were introduced in the SeismoSignal to get the accelerograms. The earthquake recording are at steps of 0.02 sec, its total length is 40.05sec and the time steps are 2000. The maximum acceleration was 0.568g at direction Y and 0.514g at direction X. The magnitude was M=6.69 and the modal damping is equal to 5%. Below the earthquake accelerogram at direction +X is presented.

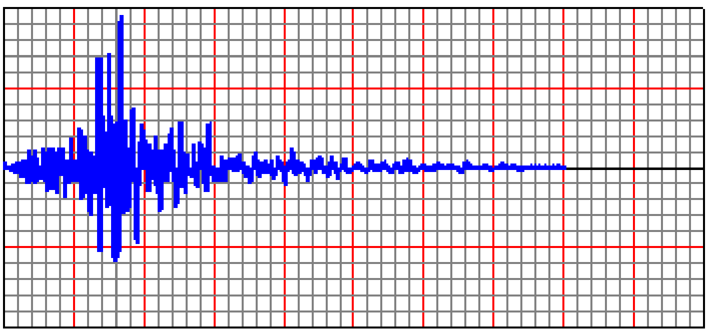

Figure 6.19: The accelerogram of the earthquake of Northridge at X direction

The structure model before and after retrofitting subjected to load time history and their maximum displacements and stresses have been plotted for the two models. The load time history used was from the Northridge earthquake. So the results can be compared before and after the strengthening. These analyses were performed in order to observe the resistant of the original building in such an earthquake and if it is going to collapse or not.

6.5.2 The procedure in SAP2000

A nonlinear direct – integration time – history analysis can be initiated from zero initial conditions (unloaded structure) or continued from a nonlinear static analysis or another direct – integration time – history nonlinear analysis. The vertical loads that correspond to dead loads and 30% of live loads with geometric nonlinearity is taken as the previous analysis case. For the analysis of undamaged bridge structures, nonlinear time history analysis is conducted including only the effects of gravity loads. The figure underneath illustrates the load combination for non linear time history analysis. The parameters for the load combinations for non linear time history analysis are the following:

- Load case type: Time history
- Analysis type: Non linear
- Time history type: Direct Integration
- Continue from state at End of nonlinear case: G+0.3Q (non linear)
- Loads applied: Function Northridge (Ex+0.3Ey) with scale factor 9.81 for X direction and 2.94 for Y direction because the accelerogram is in units of g.
- Number of output time step: 2000 (The same with the time step of the accelerogram)

- Output time step: 0.02 (The same with the time step of the accelerogram)

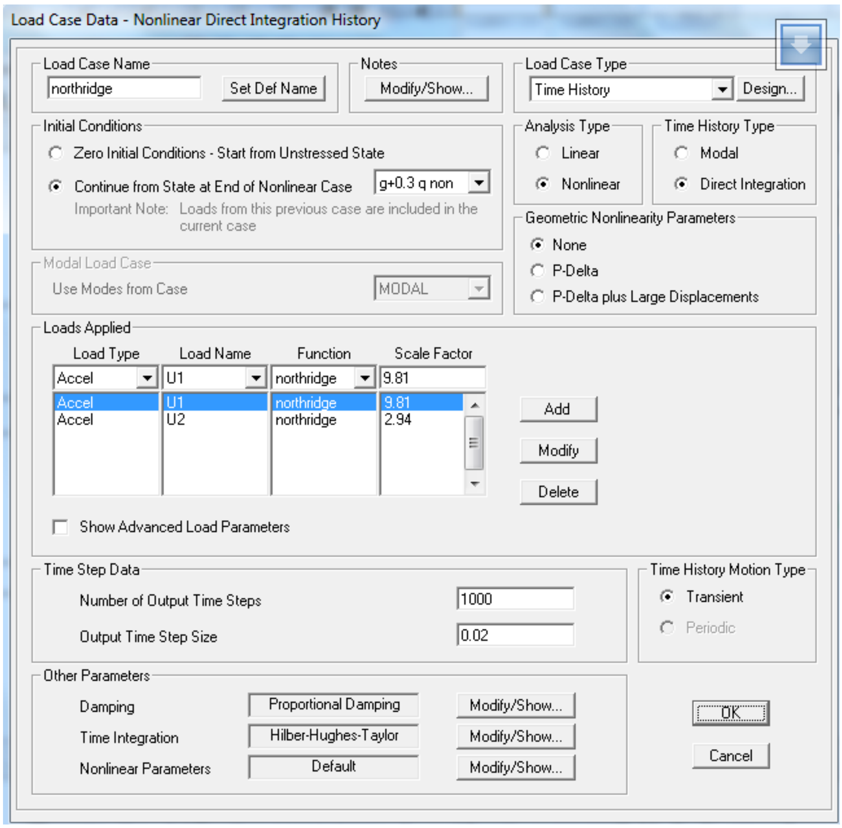

Figure 6.20: The nonlinear direct-integration time-history introduced in the program

6.5.2.1 Damping

The damping in direct – integration time – history analysis is modeled using a full damping matrix including cross coupling modal damping terms obtained from the next two sources:

1. Proportional damping from the analysis case: Damping matrix applied to the entire building calculated as a linear combination of the stiffness and mass matrices. Stiffness and mass proportional damping coefficients are specified directly or by equivalent fractions of critical modal damping at the first two modal periods. The stiffness proportional damping is linearly proportional to frequency and is related to the deformations within the construction. It can excessively damp out low period components of the oscillation. Stiffness proportional damping uses the current, tangent stiffness of the structure at each time step. Therefore, a yielding element has less damping than an elastic element, and a gap element has stiffness-proportional damping when it is closed. Mass proportional damping is linearly proportional to period. It is related to the motion of the structure and can excessively damp out long period components.

2. Proportional damping from the materials: Stiffness and mass proportional damping coefficients can be specified for individual materials. Larger coefficients can be used for soil materials than for steel or concrete. For linear direct – integration time – history analysis, the linear effective damping for the Link/Support elements is also used.

The next figure (Figure 6.22) illustrates the mass and stiffness proportional damping that was introduced in the program. The parameters for the damping are the following:

- Damping coefficient: Specify Damping by period
- First Period: 1.104 (The fundamental period of the structure)
- Second Period: 0.02 (The time step of the accelerogram)
- Damping: 0.05

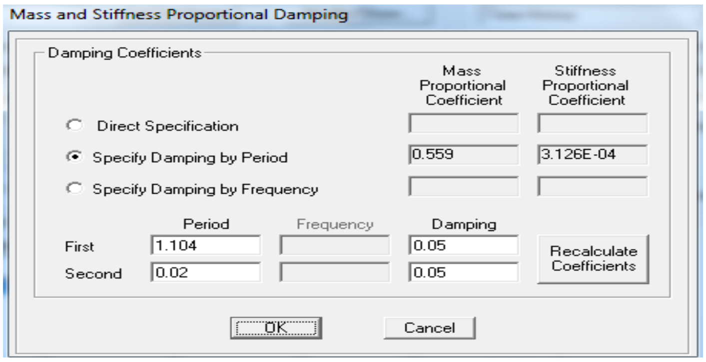

Figure 6.21: The damping parameters for non linear time history analysis

6.5.2.2 Time Integration Methods and Parameters

The same time – integration parameters and considerations are available for linear and nonlinear time history analysis. Direct integration results are extremely sensitive to time – step size, and therefore the analysis should be repeated with decreasing time – step until convergence. The time – integration methods available in SAP2000 include the Newmark's family of methods, Wilson, HHT, Collocation, and Chung and Hulbert. Newmark's average acceleration or HHT methods shall be used for seismic analysis. If the nonlinear analysis is having trouble converging, it may needed to use the HHT method with alpha = -1/3 to get an initial solution (Figure 6.22). After that it has to be re – run the analysis with decreasing the time step sizes and alpha values. These will give more accurate results.

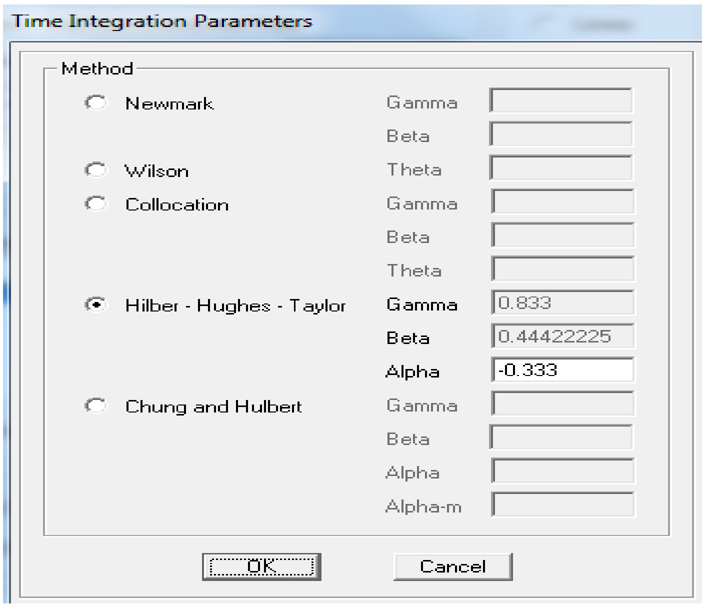

Figure 6.22: Time integration parameters with method Hilber –Hughes – Taylor

6.5.3 Results for nonlinear time history analysis

As a result a significant amount of time to solve structural systems with just a few hundred degrees of freedom is usually required. In addition, artificial or numerical damping must be added to the most incremental solution methods in order to obtain stable solutions. Due to the numerical model that accounts directly for effects of material and geometric inelastic response, the calculated deformations and internal forces are only reasonable approximations of those expected during the applied earthquake motion. For this reason, engineers must be very careful in the interpretation of the results. The results of the analysis can be checked using the applicable acceptance criteria of ATC – 32. The calculated displacements and internal forces are compared directly with the acceptance values for the applicable performance level.

Dynamic analysis may be defined simply as time – varying analysis; thus a dynamic load is any load whose magnitude, direction, and/or position varies this time. Similarly, the structural response to a dynamic load, i.e. the resulting stresses and deflections, is also time – varying, or dynamic. Static – loading condition may be looked upon as a special form of dynamic loading. Dynamic analysis is basically divided into two approaches for evaluating structural response to dynamic loads: deterministic and non – deterministic. The maximum displacements and stresses varies with time have been plotted for the two models before and after retrofitting.

The graph below (Figure 6.23) illustrates the base shear force Y varies with time before and after retrofitting. The blue color presents the base shear Y for the initial design and the red color is the strengthening structure.

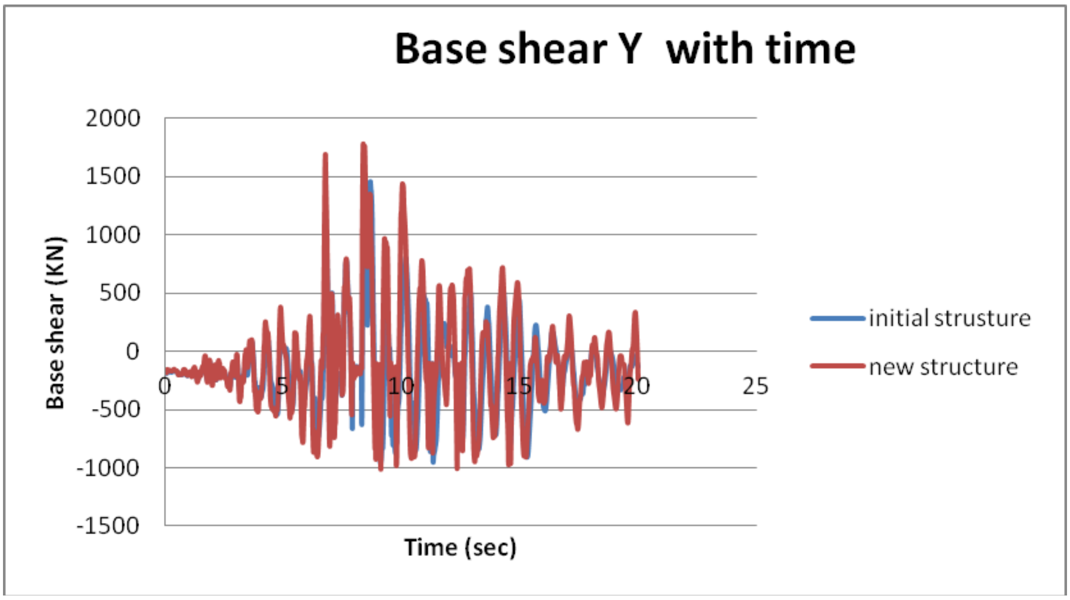

Figure 6.23: The base shear force Y varies with time

The maximum base shear force Y for the initial structure is 1454.84KN and for the retrofitted structure the maximum base shear is 1779.90KN. Comparing the initial with the new structure can be concluded that the latest one can sustain about 20% more shear forces in the base. As a result the behavior of the building due to retrofitted sections is improved.

The figure below shows how the maximum displacement varies with time before and after retrofitting. The maximum displacement is for the joint 452, which is located at the top of the structure, in the direction Y. The blue color displays the displacement for the first design and the red color is the strengthened building.

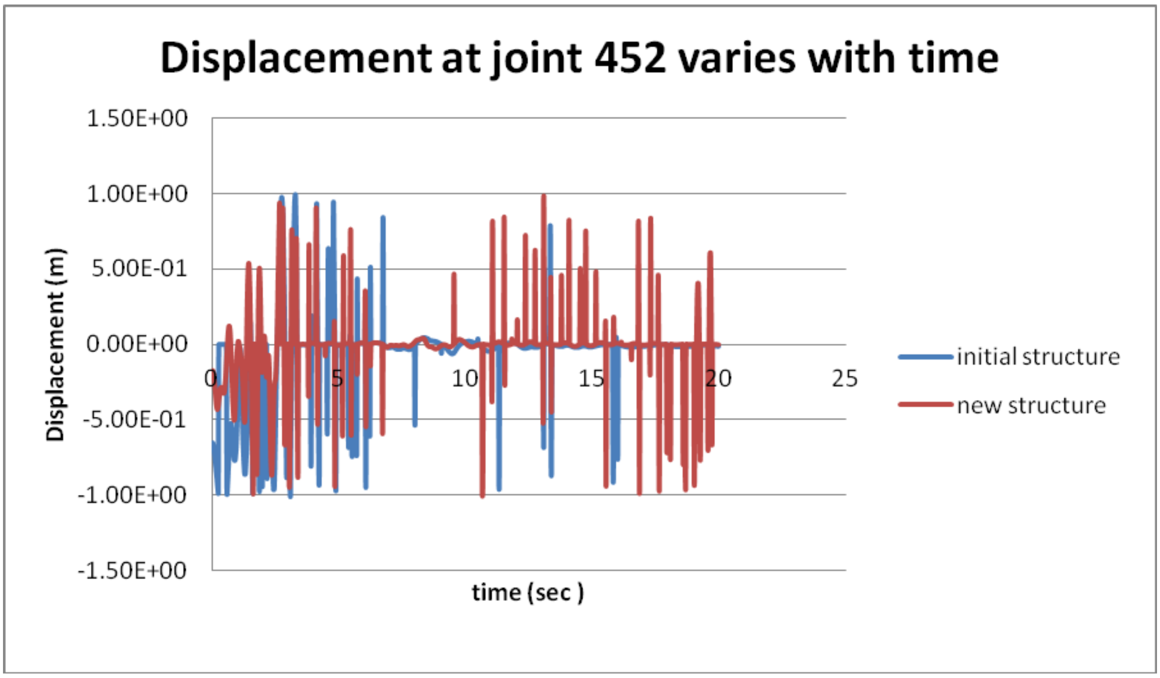

Figure6.24: Displacement at joint 452 varies with time before and after retrofitting

Comparing the two buildings it can be seen that the displacement of the strengthened one is less by approximately 10 cm. The following table illustrates the maximum displacement at joint 452 for all the case studies.

Load cases	**Maximum displacement (m) at joint 452**
Non linear static analysis +X	0.340
Non linear static analysis +Y	0.120
Non linear static analysis +X increasing reinforcement	0.319
Non linear static analysis +Y increasing reinforcement	0.100
Non linear static analysis +X retrofitted structure	0.300
Non linear static analysis +Y retrofitted structure	0.085
Non linear time history analysis initial structure	0.990
Non linear time history analysis retrofitted structure	0.890

Table 6.15: The maximum displacement at joint 452 for all the case studies

From the previous table, it is evident that the maximum displacement of the retrofitted building with non linear time history analysis is less than the corresponding displacement of the original construction. The maximum displacement of it for all the case studies is within the acceptable limits and it is not exceeding the limit of level <<LIFE SAFETY >>. Generally the behavior of the structure due to retrofitted sections is increased.

Furthermore the moment rotation curve for the repairing beam which was considered as non adequate in the previous analysis is presented. The figure underneath corresponds to the frame 152 and the joint 75 for the moment M3 and the rotation Rz.

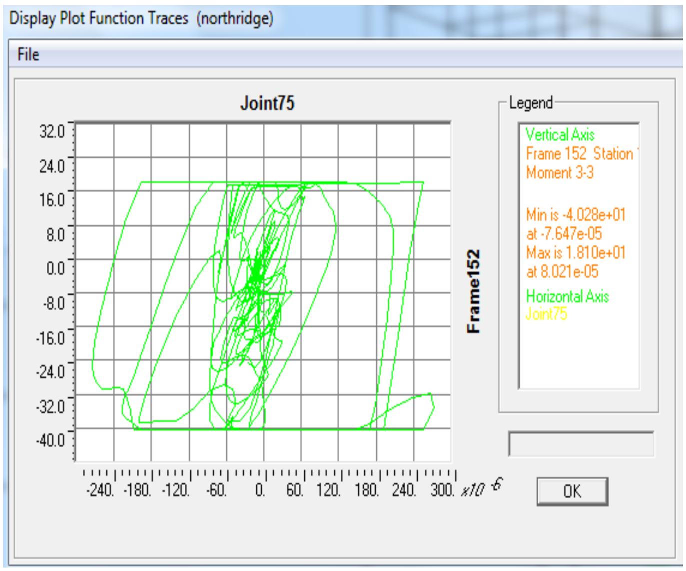

Figure6.25: Moment M3 and rotation Rz for the frame152

The following figures illustrate a comparison between the first and the second design for the overall yielding pattern of the structure at the time 19.72. It is observed that the number of elements were entered in the plastic zone have been reduced appreciably. At the new structure there are no elements that exceed the limit of level <<LIFE SAFETY >>.

- Initial structure Retrofitted structure

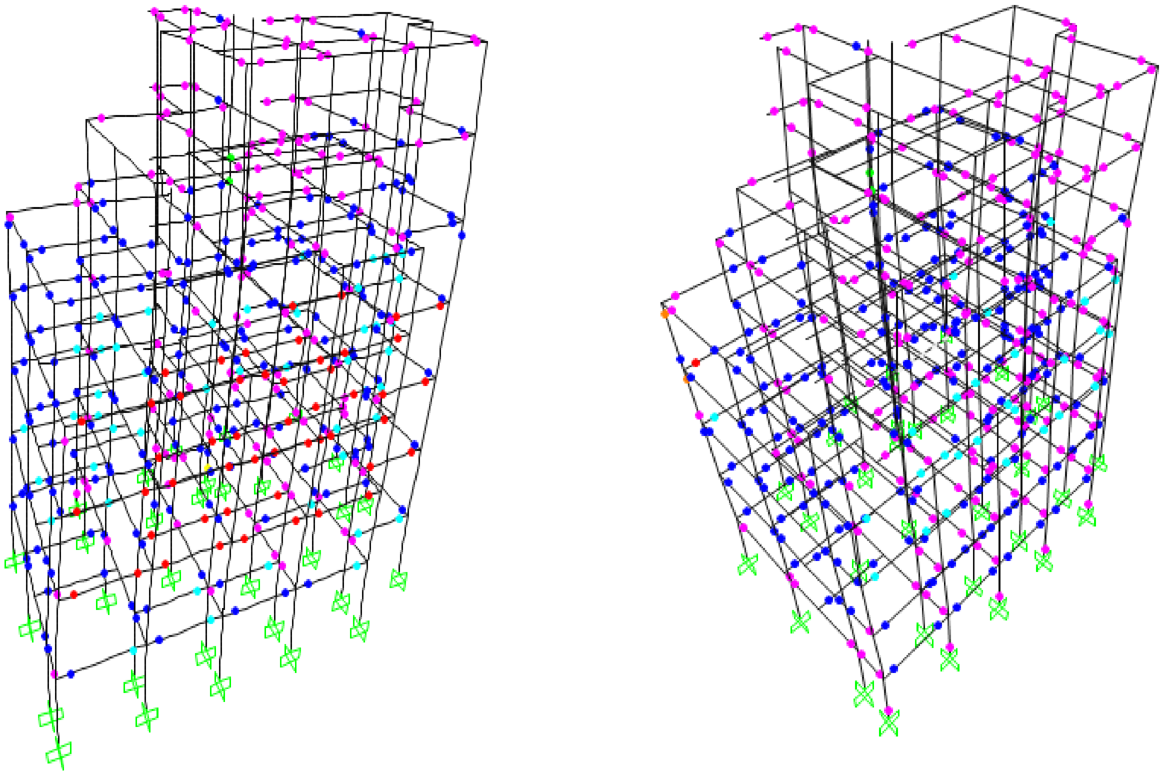

Figure 6.26: The overall yielding image structure at the time 19.72 for the initial and new structure

7. Conclusion

The current master thesis analyses the seismic behavior of an existing multistory building. Linear as well as nonlinear analysis were used for the capacity assessment of the structure. Additionally, different reinforcing scenarios were developed so as to improve the structure's behavior under an earthquake.

The studied building was designed in 1978, before modern earthquake analysis techniques were included in the Cyprus National Building Code. As a result of the inefficient design codes, the structure lacks in ductility and strength resistance. Major deficiencies of the structure are the absence of concrete walls and the existence of small column sections, beam-to-beam connections and planted columns. However, in critical areas (beam-to-column connections), additional reinforcement was placed, which explains the structure's relatively satisfactory behavior.

Linear and nonlinear analyses were used for the capacity assessment of the construction. These two different techniques were compared giving an insight to the pros and cons of each method. In this thesis linear methods that were applied to the structure are analyzed. Modal analysis results, modal response spectrum and linear time history analysis methods are presented. The results from each technique are compared in order to acquire the differences among the analysis methods.

For the assessment of the structure's carrying capacity, linear and nonlinear methods of analysis were used. Linear time history analysis provided the most conservative results compared to the other linear methods used. For the nonlinear static analysis vertical distributions of the lateral loads was applied. The most conservative results were derived by the uniform load distribution, which proved that some elements of the structure failed under the Collapse Prevention (CP) limit state.

Accordingly, two different reinforcing scenarios of the building were analyzed. The first attempt for the improvement of the structure's behavior was made by increasing the number of steel reinforcing bars in some of the columns sections, but no major change was observed. For the second scenario, concrete jackets of 10.0 cm thickness were added to some of the column and beam sections. Structural stiffness was increased in both directions and the building's maximum displacement was decreased. The overall structural behavior was improved from "CP" to a state of "LS".

Comparing the obtained results (capacity curves) in the two different configurations (existing and retrofitted building) one can say that the capacity of the retrofitted frames in terms of base shear is undoubtedly higher than the capacity of the existing frames, as expected. In particular, looking at the results obtained from the verifications of the retrofitted structure, it is observed that:

- Almost all the elements are verified to ductile and brittle mechanisms;
- For those elements which are not verified, the difference between the capacity and the demand of the element is very low; it is believed that minor improvements in the modeling of the retrofitted structure would actually eliminate these cases.

The issue of retrofitting of buildings is a quite complex. It concerns many aspects both technical and economical. There are controversies concerning when to make retrofitting a requirement, to what level must existing building be expected to perform, who should pay for rehabilitation costs, and what are the best techniques for retrofitting. It will be a long time before these questions will be answered, and in many cases it will have to be decided on case by case basis. Atc-40 has established a very useful set of guidelines for the assessment and design of seismic rehabilitation for buildings but it is still up to society and the owner to determine when and how to implement them. The only certainty is that earthquakes will continue to occur. It is better to be prepared beforehand than to suffer massive losses both in terms of lives and economic losses, in the case of an intense seismic event.

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