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M.SC. IN ANALYSIS AND DESIGN OF EARTHQUAKE RESISTANT STRUCTURES

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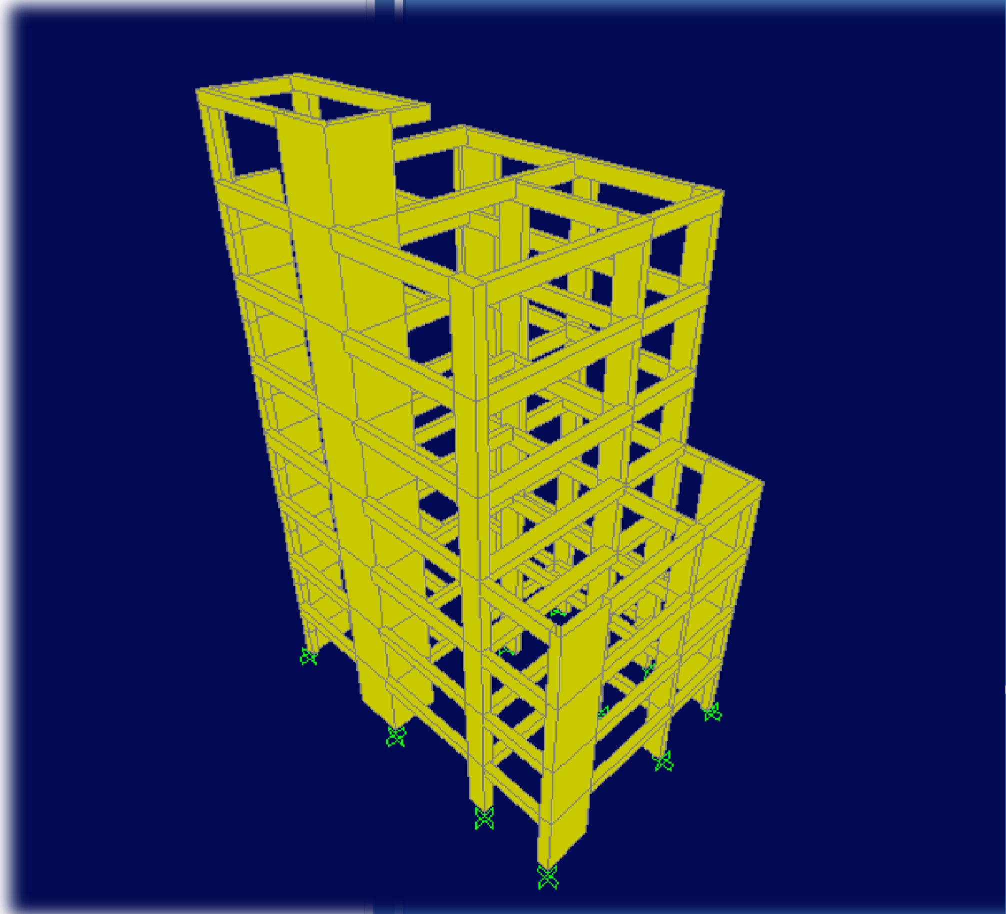
Postgraduate Thesis

Pushover Analysis & Seismic Rehabilitation of a Reinforced Concrete Structure



Submitted by:

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October 2012

Athens, Greece

Under the supervision of:

Professor Vlasis Koumousis (N.T.U.A.)

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Τίτλος: "Push-over analysis of a reinforced concrete structure"

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Για την διαμόρφωση του τελικού βαθμού λήφθηκαν υπόψη οι ακόλουθοι επιμέρους βαθμοί με τα αντίστοιχα κριτήρια βαρύτητας:
α) Για την επιστημονική επάρκεια, πληρότητα και πρωτοτυπία (50%): 10 (Βαθμός)
β) Για την εφαρμογή σε προβλήματα δομοστατικού σχεδιασμού (30%): 10 (Βαθμός)
γ) Για την παρουσίαση (20%): 10 (Βαθμός)

Τελικός βαθμός: 10

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(υπογραφές)

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1) Abstract

Pushover Analysis & Seismic Rehabilitation of a Reinforced Concrete Structure

A thesis submitted in the partial fulfillment of requirement for the award of the degree of

M.Sc in Analysis and Design of Earthquake Resistant Structures

Submitted by: Babak Momeni

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Under the supervision of:

Professor Vlasis Koumousis (N.T.U.A.)

October 2012

Athens, Greece

Engineers are constantly searching for new and more realistic methods to account for the structural behavior. Performance based strategies need to estimate the inelastic deformation and the associated damage in structures but elastic analysis cannot provide this information. Nonlinear dynamic response history analysis can provide this information, but it is a tedious procedure based on uncertainties coming from the excitation. So the scientists proposed some new design methods and rehabilitation strategies that incorporate performance based engineering concepts. It is clear that damage control should be considered as a more explicit design consideration. This goal can be reached only by consideration of some kind of non linear analysis into the seismic design methods. The most logical approach seems to be a mixture of the nonlinear static analysis (pushover analysis) and the response spectrum method.

The static pushover analysis procedure is becoming the dominant method implemented in the computer to evaluate the seismic performance of structures. The method assumes that the response of the structure can be checked by considering its first mode, and this mode during a monotonic increase of loading governs the motion constantly. There are some methods that are based on this methodology such as the capacity spectrum method (in ATC 40) and the nonlinear static procedure (in FEMA 273). The second procedure is used in ATC 40 by "displacement coefficient method" as an alternative method.

The Capacity Spectrum Method (CSM) approach is used to compare the structure's capacity with the demands of earthquake ground motion on the structure. A nonlinear force displacement curve is used to represent the capacity of the structure (pushover curve). Using the coefficients that represent effective modal masses and modal participation factors, the base shear forces should be converted to equivalent spectral accelerations and the roof displacements should be converted to equivalent spectral displacements. These spectral values define the capacity spectrum. The earthquake ground motion demands can be represented by response spectra that correspond to the level of equivalent viscous damping representing the dissipated hysteretic energy. Finally both of the curves are drawn within a same graph to determine the intersection point of the two curves that expresses the performance of the structure to the design earthquake incorporated in the particular spectrum.

In this work a simplified method for nonlinear static analysis of building structures subjected to monotonically increasing horizontal loading (pushover analysis) is presented using the SAP 2000 software. Following a step by step analysis an approximate relationship between the global base shear and top displacement of the structure is determined. During the analysis the development of plastic hinges, at different stages, throughout the building are monitored. The mathematical model, the base shear- top displacement relationships and the step by step computational procedure are described. The method is applied for the analysis of an existing seven-story reinforced concrete building. The results are presented and the evaluation of the building performance is discussed. Finally an appropriate type of intervention is proposed to improve further the seismic behavior of the building.

2) Spectral Analysis (Response spectrum seismic loading):

2-1) Introduction:

Linear elastic time history analysis results specific outputs such as member forces and time history response of joint displacements. This method has two disadvantages:

1) It gives a large amount of output information that requires lots of time for computation and doing all of the design checks.
2) The analysis should be repeated for a number of different earthquake motions to make sure that different frequency contents are considered.**(26- Csiberkeley.com)**

For the first time, the response spectrum method (RSM) was published by Maurice Anthony Biot at Caltech in his doctoral thesis. It is a strategy to find earthquake response of structures using vibrational mode shapes. He used the theories of Rayleigh for acoustics to develop the mathematical method for n degree of freedom vibrational systems. He mentioned that a building has a specific number of normal modes of vibration, and each of them corresponds to a certain frequency. He used the Fourier amplitude spectrum to find the maximum amplitude of motion of a system: the sum of amplitudes for each separate mode of vibration.

The "response spectrum" was used as a design concept in the middle of the 20^{th} century, for example in building codes in California after strong motion accelerograms became popular, the designers started to use it in earthquake engineering in the 1970s as the primary theoretical tool.

Using given masses, stiffness values, and dimensions of each storey the mathematical model of a building can be created, and then for checking of the structure's behavior the earthquake acceleration records can be applied. System response can be indicated by the linear superposition of single degree of freedom systems for various mode shapes and corresponding natural frequencies. **(20-Rebecca L. Johnson)**

Using the response spectrum method of seismic analysis for estimation of displacements and member forces in structural elements has some advantages. Using smooth design spectra (in fact they are the average of several earthquake motions), the maximum values of the displacements and member forces in each mode can be calculated with this method. **(26- Csiberkeley.com)**

2-2) Definition of a Response Spectrum:

A simple machine will be used to simulate the desired analysis. This simulator consists of a mass, m, spring with constant k, and a damper with viscous damping constant, c (with units of force x time per length):

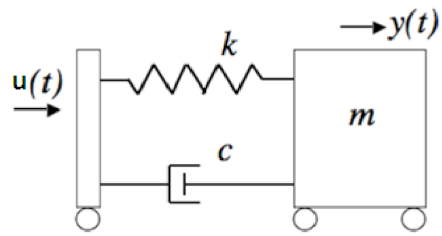

Figure 2-1- Seismic motion simulator

With the help of the Newton's second law, the response of the system to a given ground motion can be determined. The system responds to a ground displacement u(t) with absolute displacement y(t):

$$m(\ddot{y} + \ddot{u}) = -ky - c\dot{y} \tag{2.1}$$

or using natural, undamped, radial frequency, ω, and damping ratio, $\xi = \frac{c}{2\sqrt{km}}$

$$\ddot{y} + 2\xi\omega\dot{y} + \omega^2 y = -\ddot{u}$$ **(20-Rebecca L. Johnson)** (2.2)

For three dimensional seismic motion, the above equation is rewritten as:

$$\ddot{y}(t)_n + 2\zeta_n\omega_n\dot{y}(t)_n + \omega_n^2 y(t)_n = p_{nx}\ddot{u}(t)_{gx} + p_{ny}\ddot{u}(t)_{gy} + p_{nz}\ddot{u}(t)_{gz} \tag{2.3}$$

Where the three Mode Participation Factors are defined by $P_{ni} = -\emptyset_n^T Mi$ in which i is equal to x, y or z.

Two problems appear when we want to obtain the approximate response spectrum solution for this equation:

1- For each direction of ground motion maximum peak forces and displacements must be estimated.

2- After the response for the three orthogonal directions is solved, the maximum response due to the three components of earthquake motion acting at the same time should be estimated too.

The modal combination issue due to one component of motion will be explained below. For one direction, the above equation can be written as:

$$\ddot{y}(t)_n + 2\zeta_n \omega_n \dot{y}(t)_n + \omega_n^2 y(t)_n = p_{ni} \ddot{u}(t)_g \tag{2.4}$$

For solution of the this equation, with different values of ω, we should have a specific ground motion $\ddot{u}(t)g$, damping value and also we should assume that $P_{ni} = -1.0$, then we can plot a curve of the maximum peak response $y(\omega)_{Max}$. For this acceleration input, the curve is called the “displacement response spectrum” for the earthquake motion. A different curve can be drawn for each different value of damping. A plot of $\omega y(\omega)_{Max}$ is called as the “pseudo velocity spectrum” and a plot of $\omega^2 y(\omega)_{Max}$ is called as the “pseudo acceleration spectrum”. These three curves are normally plotted as one curve on special log paper. However, these pseudo values have minor significance and are not a main part of a response spectrum analysis. The real values for maximum velocity and acceleration should be calculated from the solution of this equation.

However there is a mathematical relationship between the pseudo acceleration spectrum and the total acceleration spectrum. The total acceleration of the unit mass with a single degree of freedom system, governed by the last equation, is given by:

$$\ddot{u}(t)_T = \ddot{y}(t) + \ddot{u}(t)_g \tag{2.5}$$

By solution of the last two equations at a same time we can have this equation:

$$\ddot{u}(t)_T = -\omega^2 y(t) - 2\xi\omega\dot{y}(t) \tag{2.6}$$

Therefore, for the specific case of zero damping, the total acceleration of the system is equal to $\omega^2 y(t)$. For this reason, the “displacement response spectrum” curve is usually not plotted as modal displacement $y(\omega)_{Max}$ vs. ω. It is better to present the curve in terms of S (ω) vs. a period T in seconds. Where:

$$S(\omega)_a = \omega^2 y(\omega)_{MAX} \quad \text{and} \quad T = \frac{2\pi}{\omega} \tag{2.7}$$

The pseudo acceleration spectrum, $S(\omega)_a$, curve has the units of acceleration vs. period and it has some physical significance for zero damping only. It is clear that all response spectrum curves are not a function of the properties of the structural system and they just represent the properties of the earthquake at a specific site. After estimation of the linear viscous damping properties of the structure, a proper response spectrum curve is selected. **(7-Chopra AK) (24- Steven L. Kramer) (26- Csiberkeley.com)**

2-3) Calculation of the Modal Response:

For a structural model, the maximum modal displacement can be calculated for a typical mode n with period T_n and corresponding spectrum response value (ω_n) . The maximum modal response associated with period T_n is given by:

$$y(T_n)_{MAX} = \frac{S(\omega_n)}{\omega_n^2} \tag{2.8}$$

The maximum modal displacement response of the structural model is calculated from following equation:

$$\mathbf{u}_n = y(T_n)_{MAX}\,\phi_n \tag{2.9}$$

The related internal modal forces, f_{kn} , can be calculated from standard matrix structural analysis using the same equations as needed in static analysis. **(3- Amr S. Elnashai, Luigi Di Sarno)**

2-4) Typical Response Spectrum Curves:

As an example, a ten second part of the Loma Prieta earthquake motions, recorded on a soft site in the San Francisco Bay Area, is shown in the next figure. For this earthquake, the response spectrum curves for displacement and pseudo acceleration are illustrated in the next following figures:

Typical Earthquake Ground Acceleration - Percent of Gravity:

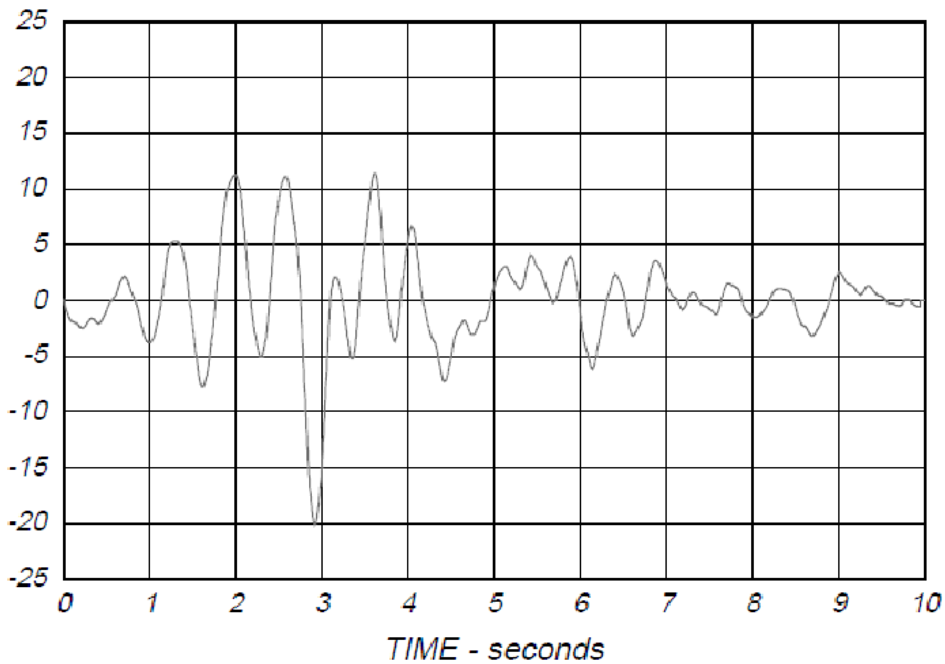

Figure 2-2- Typical Earthquake Ground Acceleration

Typical Earthquake Ground Displacements – Inches:

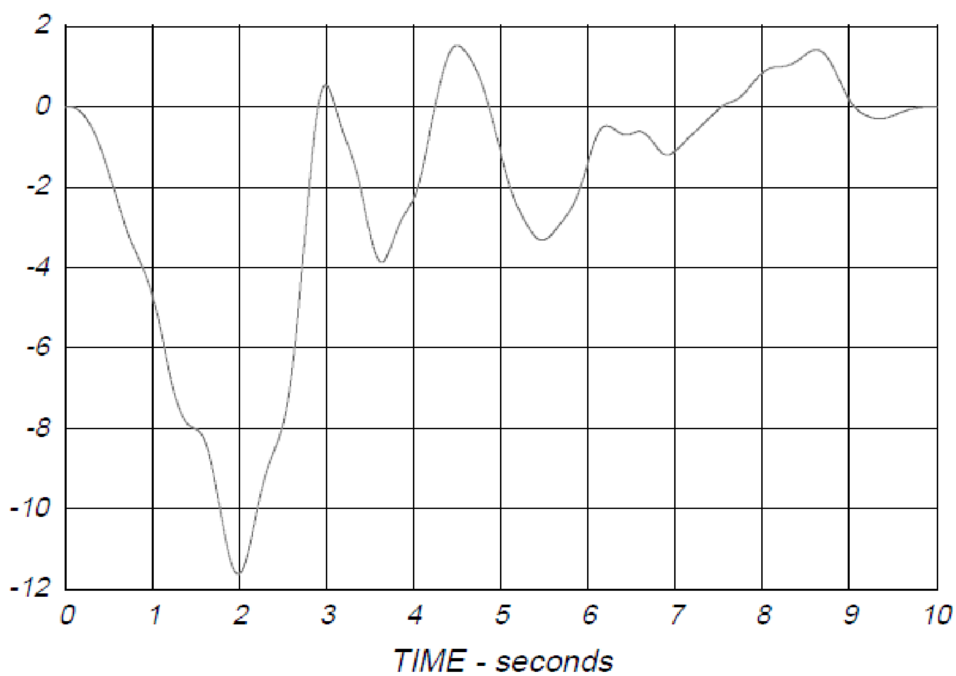

Figure 2-3- Typical Earthquake Ground Displacements

Relative Displacement Spectrum $y(\omega)_{Max}$ – Inches:

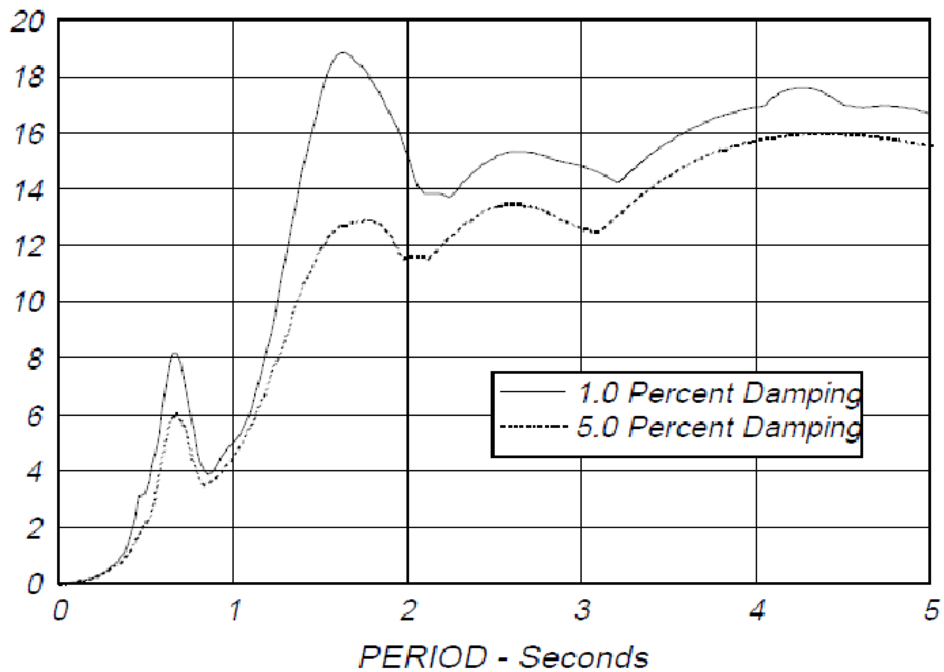

Figure 2-4- Relative Displacement Spectrum $y(\omega)_{Max}$

Pseudo Acceleration Spectrum, $S_a = \omega^2 y(\omega)_{Max}$ -Percent of Gravity:

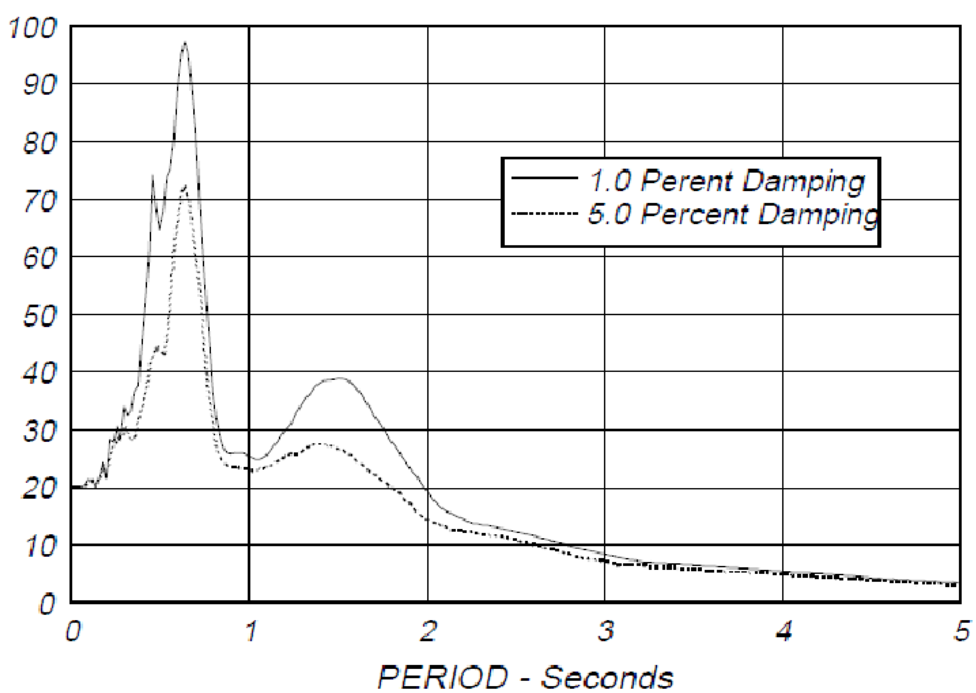

Figure 2-5- Pseudo Acceleration Spectrum, $S_a = \omega^2 y(\omega)_{Max}$

(26- Csiberkeley.com)

2-5) The CQC Method of Modal Combination:

One of the most conservative methods that can be used to calculate a peak value of displacement or force within a structure is the sum of the absolute values of the modal response. For this it should be assumed that the maximum modal values, for all modes, occur at the same time.

For calculation of the values of displacement and forces there is also another method that uses the Square Root of the Sum of the Squares, SRSS, on the maximum modal. There are some assumptions in this method but the main one is that all of the maximum modal values are statistically independent however for three dimensional structures that a large number of frequencies are close to each other, this assumption is not realistic.

The Complete Quadratic Combination (CQC) method is more adequate. It was first published in 1981. Nowadays it is used in most of the modern codes and computer software for designing and most of the engineers like to use that for seismic analysis. It is based on the random vibration theories.

For estimation of the peak value of a typical force, from the maximum modal values, by the CQC method (double summation equation), the following formula can be used:

$$F = \sqrt{\sum_n \sum_m f_n \rho_{nm} f_m} \tag{2.10}$$

Where f_n is the modal force associated with mode n. The double summation is performed for all modes. Similar equations can be used for node displacements, relative displacements and base shears and overturning moments.

The cross-modal coefficients, ρ_{nm} , for the CQC method with constant damping are

$$\rho_{nm} = \frac{8\zeta^2(1+r)\,r^{3/2}}{(1-r^2)^2 + 4\zeta^2 r(1+r)^2} \tag{2.11}$$

where $r=\omega_n/\omega_m$ and must be equal to or less than 1.0. **(3- Amr S. Elnashai, Luigi Di Sarno)**

2-6) Design Spectra:

Currently, most of the building codes use some design spectra in the following form. Design spectra are curves that represent the average spectra of many earthquakes:

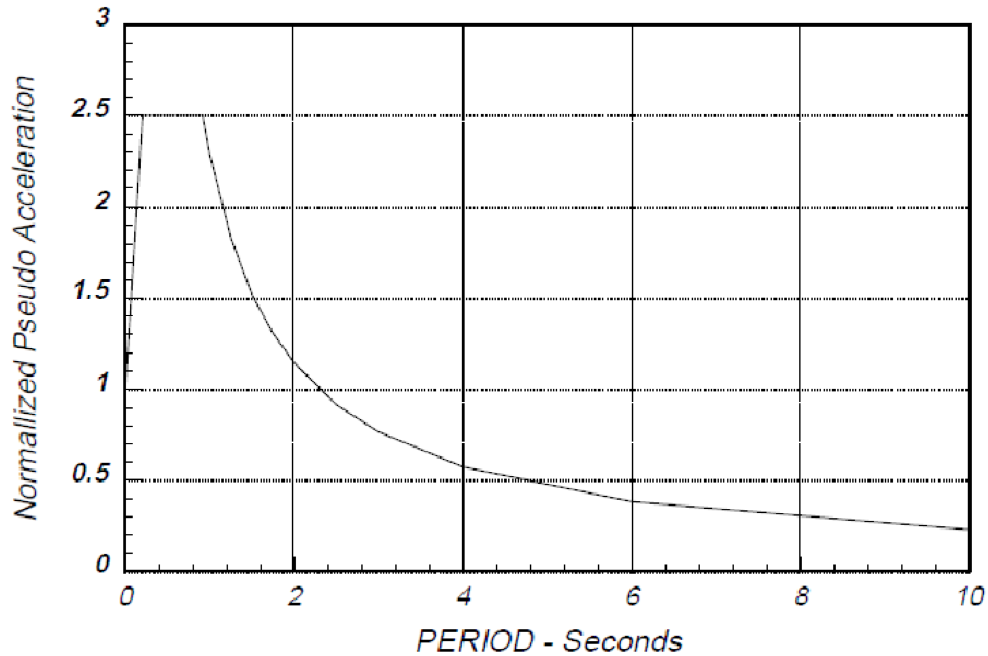

Figure 2-6- A typical Design Spectrum

Different Codes have some defined specific equations for each range of the spectrum curve and for different soil types. For important structures it is better to have a site dependent design spectrum which includes the effect of local soil conditions and distance to the nearest faults by using the attenuation relations. **(26- Csiberkeley.com)**

2-7) Direction of Earthquake in Spectral Analysis:

In fact, a structure should be designed in a way that can resist earthquake motions from all possible directions. For this purpose, some of the codes combine "100 percent of the prescribed seismic forces in one direction plus 30 percent of the prescribed forces in the perpendicular direction" to design different members while there are some other codes which require the use of 40 percent rather than 30 percent. For rectangular structures which have some clearly defined principal directions, these "percentage" rules gives approximately the same results as the SRSS method.

It is better to focus more on the direction of earthquake based on the following results:

1- All of the motions that can be created during an earthquake have one principal direction.

2- When maximum ground acceleration initiates, a principal direction exists during a finite period of time.

In fact for most of the structures that are located in areas with different geographical characteristics the main direction cannot be realized so a wise designer tries to design the structure so that it will be capable to resist an earthquake of a given magnitude from any possible direction. In addition to the motion in the principal direction, there is a probability that motions normal to that direction will occur at the same time. Also, because of the complex nature of wave propagation in three dimensional modes, it is acceptable to assume that these normal motions are statistically independent.

Because of the above mentioned assumptions the following criterion is used for designing:

- "A structure must resist a major earthquake motion of magnitude S_1 for all possible angles θ and, at the same time, resist earthquake motions of magnitude S_2 at $90°$ to the angle θ ". These motions are shown schematically in the figure below:

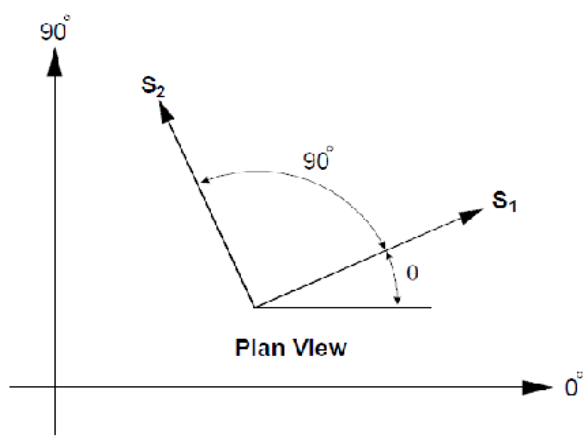

Figure 2-7- Major earthquake motions

From the above figure it is clear that the basic input spectra S_1 and S_2 are applied at an arbitrary angle θ. At some specific point in the structure, a force, stress or displacement F is produced by this input. To simplify the analysis, it will be assumed that the minor input spectrum is some fraction of the major input spectrum or:

$$S_2 = \alpha S_1 \tag{2.12}$$

Where “a” is a number between 0 and 1.0.

Recently, Menun and Der Kiureghian, presented the CQC3 method for the combination of the effects of orthogonal spectrum. The main CQC3 equation for the estimation of a peak value is:

$$F = [F_0^2 + a^2 F_{90}^2 - (1-a^2)(F_0^2 - F_{90}^2)\sin^2\theta + 2(1-a^2)F_{0-90}\sin\theta\cos\theta + F_z^2]^{\frac{1}{2}} \tag{2.13}$$

Where:

$$F_0^2 = \sum_n \sum_m f_{0n} \rho_{nm} f_{0m}$$

$$F_{90}^2 = \sum_n \sum_m f_{90n} \rho_{nm} f_{90m}$$

$$F_{0-90} = \sum_n \sum_m f_{0n} \rho_{nm} f_{90m}$$

$$F_Z^2 = \sum_n \sum_m f_{zn} \rho_{nm} f_{zm}$$

in which f_{0n} and f_{90n} are the modal values produced by 100 percent of the lateral spectrum applied at 0 and 90 degrees respectively and f_{zn} is the modal response from the vertical spectrum which can be different from the lateral spectrum.

It is important to mention that for equal spectra a =1, the value of F is not a function of θ and the selection of the analysis reference system is arbitrary or:

$$F_{MAX} = \sqrt{F_0^2 + F_{90}^2 + F_z^2} \tag{2.14}$$

This shows that it is possible to perform only one analysis, with any reference system, and then the output structure will have all members that are capable to equally resist earthquake motions from all probable directions. This method is acceptable by most building codes.

For a=1 the CQC3 method reduces to the SRSS method. It seems that this is so conservative since real ground motions of equal value in all directions have not been recorded yet. Normally, the value of θ is not known; therefore, it is necessary to calculate the critical angle that produces the maximum response. Differentiation of the fundamental CQC3 equation and setting the results to zero yields:

$$\theta_{cr} = \frac{1}{2}\tan^{-1}\left[\frac{2F_{0-90}}{F_0^2 - F_{90}^2}\right] \qquad (2.15)$$

Two roots exist for the last equation that must be checked in order that the following equation will be maximum:

$$F_{MAX} = [F_0^2 + a^2 F_{90}^2 - (1-a^2)(F_0^2 - F_{90}^2)\sin^2\theta_{cr} - 2(1-a^2)F_{0-90}\sin\theta_{cr}\cos\theta_{cr} + F_z^2]^{\frac{1}{2}} \qquad (2.16)$$

First of all a designer should try to select the desired reference system and then based on this selection, for three dimensional response spectra analyses, the design of elements for 100 percent of the prescribed seismic forces in one direction plus 30 or 40 percent of the prescribed forces applied in the perpendicular will be done. These percentage combination rules are empirical and can underestimate the design forces in some specific members and produce a member design which is relatively weak in one direction.

In the SRSS method, design forces will not be a function of the reference system because in this method a combination of two 100 percent spectra analyses with respect to any user defined orthogonal axes will be selected and this will produce design forces that are not a function of the reference system. Therefore, the resulting structural design has equal resistance to seismic motions from all directions. The CQC3 method should be used if a value of a less than 1.0 can be acceptable. It will produce realistic results that are not a function of the user selected reference system. **(26- Csiberkeley.com)**

2-8) Limitations of the Response Spectrum Method:

It is clear that use of the response spectrum method has some limitations and also it will never be an accurate method for nonlinear analysis of multi degree of freedom structures because time variable is eliminated from the dynamical problem. Some of the limitations will be mentioned in the following:

2-8-1) Story Drift Calculations:

All displacements produced by the response spectrum method are positive numbers, so a plot of a dynamic displaced shape does not contain lots of information because each displacement is an estimation of the maximum modal value. Interstory displacements are used to evaluate damage to nonstructural elements. They cannot be calculated directly from the probable peak values of displacement.

2-8-2) Estimation of Spectra Stresses in Beams:

For calculation of the stresses in a beam the following relation can be used:

$$\sigma = \frac{P}{A} + \frac{M_y x}{I_y} + \frac{M_x y}{I_x} \tag{2.17}$$

This equation can be evaluated based on the coordinates of the each specified point in the cross section and for the calculated maximum spectral axial force and moments (which are all positive values). The calculated stress is a bit conservative because at a same time all forces will probably not have their maximum values.

To have an exact response spectrum analysis, the evaluation of the above equation should be done for each mode of vibration. In this procedure we should take into consideration the relative signs of axial forces and moments in each mode. Then the exact value of the maximum stress can be calculated from the modal stresses using the CQC double summation method.

2-8-3) Design Checks for Steel and Concrete Beams:

In most of the cases, the design strength ratios are a nonlinear function of the axial forces in the members, so the ratios cannot be calculated in each mode and this is not favorable because most design check equations for steel structures are based of design strength ratios.

For concrete structures, because of the nonlinear behavior of concrete members, some new studies should be performed to complete the method for the use of maximum spectral forces in a design check equation. A time history analysis may be the only approach that will produce logical design forces.

2-8-4) Calculation of Shear Force in Bolts:

Calculating of the maximum shear force from a vector summation is not acceptable because the x and y shears do not have their maximum values at the same time. The logical way to estimate the maximum shear forces in a bolt is to check the maximum bolt shear at several different angles about the bolt axis. But this procedure is a tedious one and it is better to use computer software to do that. The same problem exists if principal stresses should be calculated from a response spectrum analysis. The maximum and minimum values of the stress at each point in the structure can be calculated by checking at several angles. **(26-Csiberkeley.com)**

3) Non-linear Static Pushover Analysis in American Code (ATC-40):

3-1) Introduction:

Using elastic analysis, the location of the first yielding can be found. It gives a good indication of the elastic capacity of the analyzed structure. Despite of all these facts, it cannot predict failure mechanisms and account for redistribution of forces during progressive yielding which are essential issues in modern design.

If someone wants to identify the modes of failure and the potential of collapse, he should use inelastic analysis procedures. Using the inelastic procedures for design and evaluation, the behavior of the structures which are subjected to major earthquakes can be monitored. (Where it is assumed that the elastic capacity of the structure will be exceeded) Although this is associated with some new uncertainties related with code and elastic procedures.

When a structure is subjected to an earthquake, some displacements (and in turn deformations) in its individual elements can be seen. In the beginning, the element deformations will be in their elastic (linear) range and no damage will occur but at the advanced levels, element deformations will be more than their linear elastic capacities and some damages will be experienced in the building.

To have a proper seismic performance, one should try to limit the lateral displacements that are initiated from the earthquake, to levels at which the damage sustained by the building's elements will be consistent to the needed performance objective and this can be achieved by having an adequate lateral force resisting system. The basic factors that affect the lateral force resisting system's ability to perform well include:

- The building's mass, stiffness, damping, and layout configuration
- The deformation capacity of its elements
- The intensity and frequency content of the earthquake it must resist. **(6-ATC-1996)**

For seismic performance evaluation of old and new structures, the static pushover analysis can be used. This type of analysis gives some output information on seismic demands imposed by the design earthquake on the structural system and its components.

The pushover analysis is a static nonlinear analysis. In this kind of analysis the structure will be loaded under permanent vertical loads and gradually increasing lateral loads that approximately simulates the earthquake forces. Then a graph of the total base shear vs. top displacement in a structure can be plotted by this analysis. By help of this plot any premature failure or weakness in the structure can be tracked. This process continues up to failure and finally the designer will be able to determine the collapse load and ductility capacity. Also the plastic rotation in the elements can be monitored, and lateral inelastic forces versus displacement response for whole of the structure can be analytically computed. By help of this kind of analysis the deficiencies of a structure can be determined and then the proper rehabilitation strategy can be selected.

Using the pushover analysis, the expected performance of structural systems can be evaluated. This can be done by derivation of the performance of the structural system by estimating its strength and deformation demands in design earthquakes by applying of static inelastic analysis, and comparing these demands to available capacities at the performance levels of interest.

The most important performance parameters that are usually evaluated are:

- ✓ global drift, interstory drift
- ✓ Inelastic element deformations (either absolute or normalized with respect to a yield value)
- ✓ Deformations between elements
- ✓ Element connection forces (for elements and connections that cannot sustain inelastic deformations).

The inelastic static pushover analysis is a method for estimation of seismic force and deformation demands. It calculates with an approximate procedure the redistribution of internal forces that no longer can be resisted within the elastic range of structural behavior of elements.

There is a critical concept in this kind of analysis and this is the “target displacement”. Performing the pushover analysis leads to estimation of the target displacement magnitude, as a representative displacement, at which seismic performance evaluation of the structure is to be valid. In fact the target displacement serves as an estimation of the global displacement of the structure is expected to experience in an expected design earthquake.

In this procedure we assume that a MDOF structure can be simulated by an equivalent SDOF system and the target displacement of the original structure can be estimated by the target displacement of the SDOF mass center. This concept is acceptable only with some limitations and only if great care is taken in incorporating in the predicted SDOF displacement demand all the important ground motion and structural response characteristics that significantly affect the maximum displacement of the MDOF structure. In this assumption the maximum MDOF displacement is controlled by a single shape factor and the higher mode effects are not considered: **(19- Pu Yang And Yayong Wang)**

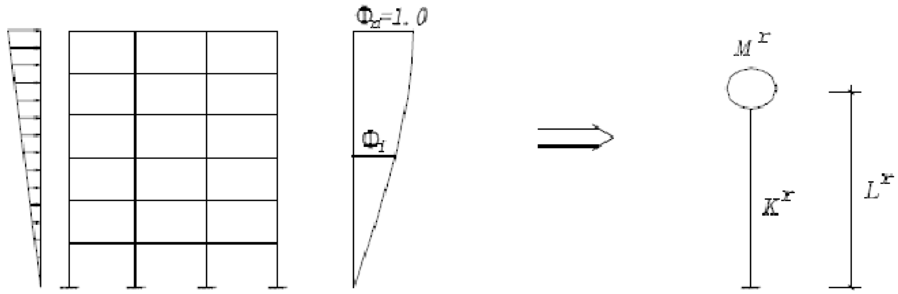

Figure 3-1- A 1st mode approximation of a MDOF system generalized SDOF system

As it was mentioned before, in this approach, the model of the building is subjected to monotonically increasing lateral forces or displacements. This procedure continues until either a target displacement is reached, or the building collapses. The target displacement represents the maximum probable displacement to be experienced during the design earthquake. Finally the capacity curve can be plotted which can be compared with design spectrum of the members and the designer can determine whether the building is safe or needs strengthening and in what extent. **(3- Amr S. Elnashai, Luigi Di Sarno) (15- M. Seifi, J. Noorzaei, M. S. Jaafar, E. Yazdan Panah) (18- Peter Fajfar, M.EERI) (27-architectjaved.com)**

3-2) Methods to perform Nonlinear Analysis:

There are two main components in this type of analysis: demand and capacity. Demand is a representation of the earthquake intensity and capacity is a representation of the structure's ability to resist applied lateral forces. There are some simplified nonlinear analysis procedures using pushover methods such as the capacity spectrum method and the displacement coefficient method. In these methods, three primary elements are needed to determine: capacity, demand (displacement) and performance. In the following all of these elements will be explained:

Capacity:

The strength and deformation capacities of the elements of a structure have a significant impact to the overall capacity of a structure. Some form of the nonlinear analysis, such as the pushover procedure, should be used to determine the capacities beyond the elastic limits. This procedure uses a series of sequential elastic analyses in a step by step procedure, appended in a way to approximate a force displacement capacity diagram of the overall structure. The mathematical model of the structure is modified to consider reduced resistance of yielding components. A lateral force distribution is again applied until additional components yield. This process is continued until the structure becomes unstable or until a predetermined limit is reached.

Demand (displacement):

Because of the ground motions during an earthquake, complex horizontal displacement patterns can be produced in structures that may vary with time and it is impractical to track this motion at every time step to determine structural design requirements. In nonlinear analysis methods it is better to use a set of lateral displacements as a design requirement. For a given structure and earthquake, the displacement demand is an estimate of the maximum expected response of the building during the earthquake.

Performance:

After plotting of the capacity and demand curves, a performance check can be done to verify that structural and nonstructural components are not damaged beyond the acceptable limits of the performance levels for the forces and displacements that are needed by the displacement demand.

In the following sections, step by step procedures to determine capacity, demand and performance will be explained.

3-2-1) Step by step procedures to determine capacity:

By calculating and plotting the pushover curve of a structure, the structure capacity can be determined. In other words structure capacity is represented by a pushover curve. There are some ways that someone can do that but the best way is tracking the base shear and the roof displacement to plot the force displacement curve. It assumes that the main mode of vibration of a structure is its first mode and based on this assumption the capacity curve is generally plotted to represent the first mode response of the structure. It should be mentioned that this assumption is generally valid for buildings with fundamental periods of vibration less than about one second so for structures with higher periods, the other modes of vibration should be considered too.

The derivation steps can be summarized as below:

1. Modeling of the structure within the software.
2. Applying a single concentrated horizontal force at the top of the structure and the gravity loads that correspond to dead loads and 30% of live loads.
3. Alternatively applying lateral forces to each story following one of the methods below:
   - In proportion to the standard code procedure without the concentrated force “F” at the top
   - In proportion to the product of story masses and first mode shape of the elastic model of the structure.
4. Same as Level 3 until the first yielding appears. For each increment beyond yielding, adjust the forces to be consistent with the changing deflected shape.
5. Same as levels 3 and 4, but include the effects of the higher modes of vibration for determination of the yielding in individual structural elements while plotting the capacity curve for the building in terms of first mode lateral forces and displacements. The higher mode effects may be determined by doing higher mode pushover analyses (modal push-over analysis, Chopra) (i.e., loads may be applied increasingly in proportion to a mode shape other than the fundamental mode shape to determine its inelastic behavior.) For the higher modes the structure is being both pushed and pulled concurrently to maintain the mode shape.
6. Calculating elemental forces for the required combinations of vertical and lateral loads.
7. Recording the base shear and the roof displacement. (It is also useful to record member forces and rotations because they will be needed for the performance check)
8. Revising the model using zero (or very small) stiffness for the yielded elements.
9. Applying a new increment of lateral load to the modified structure such that another element (or group of elements) yields. (The actual forces and rotations for elements at the beginning of an increment are equal to those at the end of the previous increment. However, each new application of an increment of lateral load needs a separate analysis which should start from zero initial conditions. Thus, for determination of the yielding time of the next element, the forces from the current analysis should be added to the sum of those from the previous increments. Also, for determination of the element rotations, the rotations from the current analysis should be added to the sum of those from the previous increments.)
10. Adding the increment of lateral loads and the corresponding increment of roof displacement to the previous totals to give the accumulated values of base shear and roof displacement.
11. Repeating steps 8, 9 and 10 until the structure reaches an ultimate limit, such as:

-Instability because of P-Δ effects

-Distortions clearly beyond the desired performance level

-An element (or group of elements) reaches a lateral deformation level at which significant strength degradation begins

-An element (or group of elements) reaches a lateral deformation level at which loss of gravity load carrying capacity appears

The below figure is a typical capacity curve:

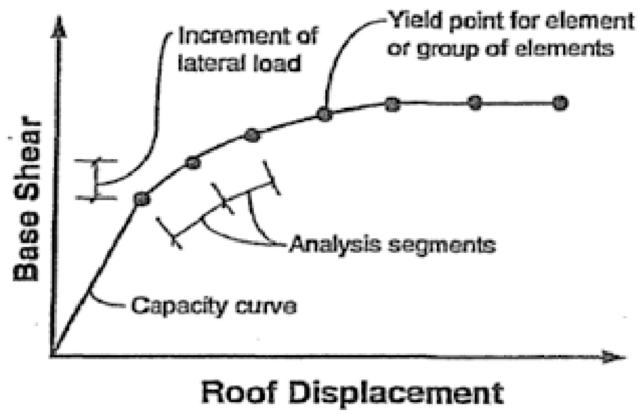

Figure 3-2- Typical capacity curve

12. If the incremental loading is stopped in step 11 because of reaching a lateral deformation level at which all or a big portion of an element's (or group of elements) load cannot be carried more (Just because its strength has significantly degraded), then the stiffness of that element(s) is reduced, or eliminated. A new capacity curve can then be plotted, starting with step 3 of this step by step process. For a complete definition of the overall loss of strength, create as many additional pushover curves as necessary. The below figure illustrates this process, for an example where three different capacity curves are required.

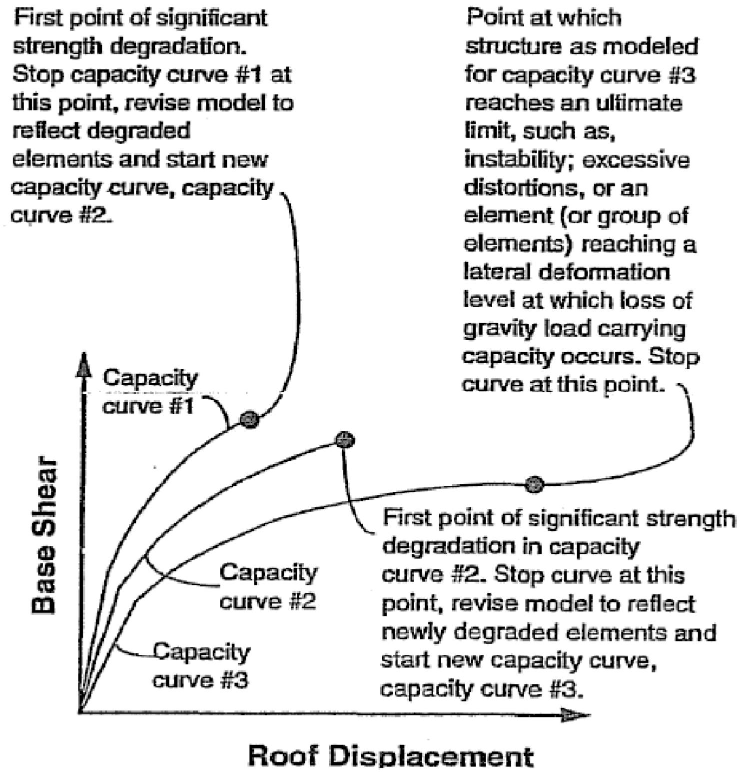

Figure 3-3- Effect of the element strength degradation in capacity curves

Now we can plot the final capacity curve. For this issue, initially follow the first curve, then a transition to the second curve at the displacement corresponding to the initial strength degradation should be done, and so on. This curve will have a "sawtooth" shape, as shown in the figure below:

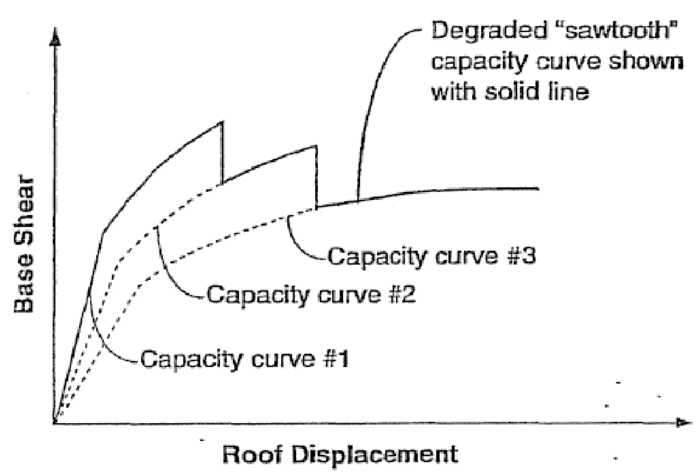

Figure 3-4- Sawtooth Capacity Curve

3-2-2) Step by step procedures to determine demand:

For a given performance level, to determine compliance with that level, a displacement which is consistent with the seismic demand, along the capacity curve must be determined. Two methods for determining this displacement will be mentioned in the following:

The capacity spectrum method is based on searching and finding a point on the capacity spectrum that also is located on the suitable demand response spectrum, modified (reduced) for nonlinear effects. The demand displacement in the capacity spectrum method is related to the performance point on the capacity spectrum which the seismic capacity of the structure is equal to the seismic demand imposed on the structure by the specified earthquake.

The second method that is used in FEMA-273 called the coefficient method. This method is based on statistical analysis of the results of time history analysis of single degree of freedom models of different types. The demand displacement in the coefficient method is called the target displacement.

Equal displacement approximation:

Applying the "equal displacement approximation" method, we can estimate the displacement related to a given seismic demand. This estimation will be based on the this assumption that the inelastic spectral displacement is same as that which would exist if the structure behaves completely elastic. In some cases, especially for the longer period ranges (T> 1.0 seconds), the "equal displacement approximation" will usually gives some similar results to the capacity spectrum and coefficient methods. In other cases, especially for the short period, ranges (T<0.5 seconds) the resulted displacements from the simple approximation may be deeply different from the results obtained using the capacity spectrum or coefficient methods. (Less than)

The equal displacement method is often a useful tool for estimation of the initial trial performance point in the iterative capacity spectrum procedures:

In the following the graphical representation of this method is illustrated:

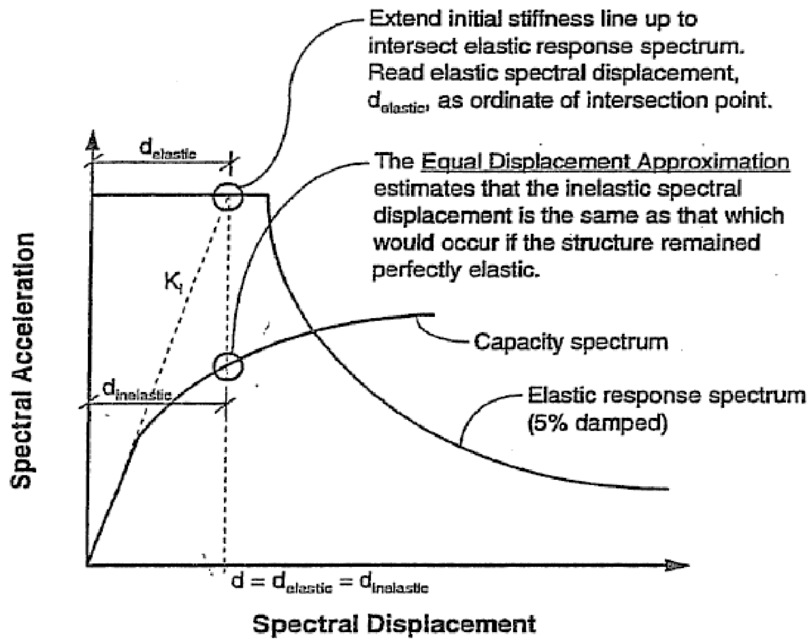

Figure 3-5- Equal Displacement Approximation

3-2-2-1) Calculating Demand using the capacity spectrum Method:

The Performance Point must fulfill the two conditions below in order to satisfy the method's requirements:

1) The point must be located on the capacity spectrum curve in order to represent the structure at a given displacement.
2) The point must be located on a spectral demand curve which is modified (reduced) from the elastic five percent damped design spectrum that represents the nonlinear demand at the same structural displacement.

In this methodology, spectral reduction factors are given in connection with the effective damping. An effective damping (approximate one) can be calculated based on the shape of the capacity curve, the estimated displacement demand and the resulting hysteresis loop. Probable deficiencies in the real building hysteresis loops, including the effects of degradation and duration, are considered by reductions in equivalent viscous damping values (that are theoretically calculated). Generally, determination of the performance point requires a trial and error searching procedure for satisfaction of the two criterions that are mentioned above.

3-2-2-2) Calculating Demand using the displacement coefficient Method:

In this method, the target displacement obtained using the displacement coefficient method will be equal to the resulted displacement from the equal displacement method modified by different coefficients. The displacement coefficient method presents a direct numerical procedure for displacement demand calculation. In this method there is no need to convert the capacity curve to the spectral coordinates. **(6-ATC-1996)**

3-3) Evaluating the structure by the pushover analysis method:

In some of the analysis methods, the seismic capacity and the seismic demand can be compared with each other to evaluate the response of the structure. Also after applying the required strengthening, the same methods can be used to investigate or verify that if the intervention is effective or not.

For derivation of the capacity spectrum, an approximate nonlinear incremental static analysis for the structure can be used and in the process of performing this incremental nonlinear static analysis, a capacity curve can be plotted for the building. In fact this capacity curve is a plot of the total lateral seismic shear force demand "V" on the structure for different increments of loading, against the lateral deflection (displacement) of the building at the roof level because of the applied lateral forces. If a building had infinite linear elastic capacity, this capacity curve would be a straight line. Then the slope of that line would be equal to the global stiffness of the structure but because real buildings do not have infinite linear elastic capacities, the capacity curve usually consists of a series of straight line segments with decreasing slope trend, representing the progressive degradation of the structural stiffness that happens just because of this fact that the building is subjected to the increased lateral displacement, yielding, and damage. If somebody draws a straight line from the origin of the plot to a point on the curve at any lateral displacement, d, the slop of this line represents the secant or effective stiffness of the structure when pushed laterally to that displacement. A typical capacity curve is shown below:

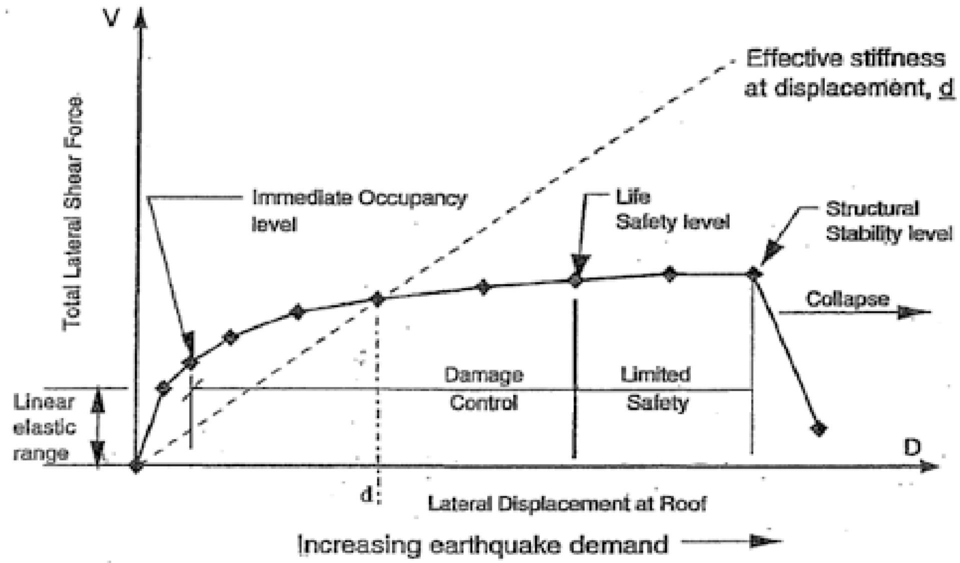

Figure 3-6- Break-down points in a typical capacity curve

In this figure, the points that are shown by the squares represent the occurrence of important events in the lateral response history of the structure. Some of them are:

-Initiation of the yield in a particular structural element or a specific type of damage, such as spalling of cover concrete on a column or shear failure of a spandrel element.

Each point is determined by a different analysis sequence. Then by evaluating the cumulative effects of damage that is initiated at each of the individual events and the overall behavior of the structure at increasing lateral displacements, it will be possible to determine some points on the capacity curve in which total structural lateral displacements that are related to the limits on the various structural performance levels are represented, as it has been done in this figure.

Usually some element deformation parameters such as the plastic chord rotation of a beam or shear angle of a wall are selected as limiting values. For these limitations, some acceptance criteria exist which are reasonable approximate estimates of the average deformations at which certain types of element behavior such as cracking, yielding, spalling, or crushing. After doing the incremental static nonlinear analyses the structure can be evaluated for the acceptance criteria by checking the cumulative deformations of all important structural elements.

At the specific points on the capacity curve, someone can see some local deformation and maybe sometimes a few elements exceeds the allowable deformation limits for a structural performance level but this does not necessarily mean that at that points the structure as a whole reaches that structural performance level. Most of the structures consist of many elements and have considerable redundancy so by having some damage to a small group of these elements; we cannot say that the whole of the structure is in an unacceptable condition concerning the overall performance of the building. For locating of the points along the capacity curve for a structure related to its different structural performance levels, the performance of the building should be analyzed as a whole and then the impact of the predicted damage for the various elements on the overall behavior of the building should be considered.

One of the concepts that should be considered in these issues is differentiating between the "primary" and "secondary" elements. Primary elements are needed as a part of the vertical and lateral force resisting system for the structure and all of the remaining elements are secondary elements. Since degradation of the secondary elements does not have a deep impact on the lateral load resisting capacity of the building, secondary elements are usually permitted to sustain more damages than primary ones. Sometimes, when the development process of the capacity curve is doing, it is obvious that a few elements fail to meet the acceptance criteria for a given performance level at an increment of lateral loading and displacement. In such kind of cases the designer can consider these nonconforming elements as secondary. This action enable us to use more liberal acceptance criteria for these few secondary elements.

By converting of the capacity curve from lateral force (V) vs. lateral displacement (d) coordinates to spectral acceleration (Sa) vs. spectral displacement (Sd) coordinates, the capacity spectrum curve for the structure can be derived. This can be done by using the modal shape vectors, participation factors and modal masses (which are obtained from a modal analysis of the structure). In the capacity curve with the Sa vs. Sd coordinates, we can draw some radial lines from the origin of the plot through the curve at different spectral displacements. These lines have a slope of $(\omega')^2$, where ω' is the radial frequency of the effective (or secant) first mode response of the structure (if the structure is subjected to an earthquake to reach to that spectral displacement). For calculation of the effective period of the structure for each of these radial lines, the equation $T' = 2\pi/\omega'$ can be used (if it is pushed to given spectral displacements).

In the capacity spectrum method, using an elastic response spectrum, the seismic demand can be determined at the first step. This spectrum is plotted in spectral ordinates format representing the spectral acceleration as a function of spectral displacement. With this procedure the demand spectrum can be overlaid on the capacity spectrum for the building. If the intersection of the demand and capacity spectra will be located in the linear limits of the capacity, this would define the real displacement for the structure but usually the point is not in the linear limits of the capacity and some inelastic nonlinear behavior can be seen during analyses too.

To find the point where demand and capacity are equal, the designer selects a point on the capacity spectrum as an initial estimate and then with the spectral acceleration and displacement defined by this point, the engineer then can calculate reduction factors to apply to the 5% elastic spectrum to consider the hysteretic energy dissipation, or effective damping, associated with the specific point. If the reduced demand spectrum intersects the capacity spectrum at or near the initial assumed point, then it is the solution for the unique point where capacity equals demand. When the performance point has been found, the designer can check the acceptability of a rehabilitation strategy to see if it meets the project higher performance objectives or not. This can be checked by evaluating where the performance point falls on the capacity curve. With this information, the effectiveness of different rehabilitation strategies to achieve the project performance objectives can be assessed and one of them can be selected.

3-4) Building performance levels and ranges:

In below, the definition of the building performance levels and ranges will be elaborated:

Performance level:

The potential condition of a building after an earthquake; a well defined point on a scale measuring how much loss is caused by earthquake. (This loss may be in terms of property, operational capability or even casualties)

Performance range:

This is defined as a band of performance, rather than a discrete performance level.

Designations of performance levels and ranges:

Each Performance level has two separated concepts to be described: damage of structural and damage of nonstructural systems; structural designations are S-1 through S-5 and nonstructural designations are N-A through N-D.

Building performance level:

To have a complete description of an overall damage level, a combination of a structural performance level and a nonstructural performance level is needed. There are various methods and design criteria to achieve several different levels and ranges of seismic performance. Four Building Performance Levels are ***Collapse Prevention, Life Safety, Immediate Occupancy, and Operational***. Each level is related to a discrete point on a continuous scale which describes the building's expected performance. (Or alternatively how much damage, economic loss and disruption may occur).

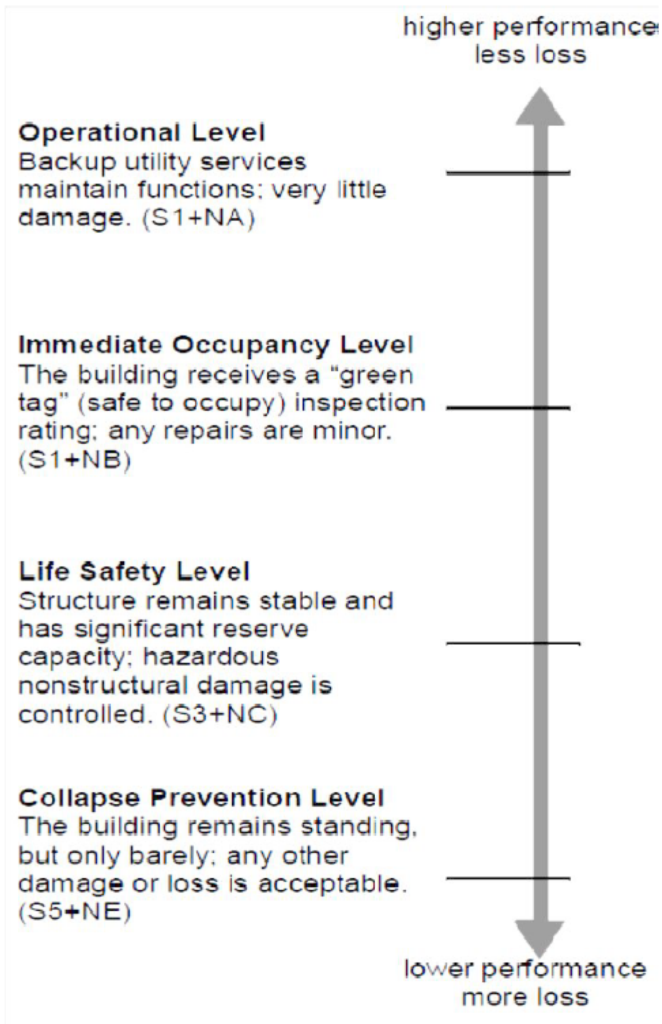

Figure 3-7- Building performance levels

Performance levels consist of a structural performance level that describes the limiting damage state of the structural systems and a nonstructural performance level that describes the limiting damage state of the nonstructural systems. Using three structural performance levels and four nonstructural performance levels, all of the four basic building performance levels can be formed.

The three Structural Performance Levels and two Structural Performance Ranges consist of:

•S-1: Immediate Occupancy Performance Level

•S-2: Damage Control Performance Range (extends between Life Safety and Immediate Occupancy Performance Levels)

•S-3: Life Safety Performance Level

•S-4: Limited Safety Performance Range (extends between Life Safety and Collapse Prevention Performance Levels)

• S-5: Collapse Prevention Performance Level

In addition, there is the designation of S-6, structural performance not considered, for the situation where only nonstructural improvements are made.

The four Nonstructural Performance Levels are:

• N-A: Operational Performance Level

• N-B: Immediate Occupancy Performance Level

• N-C: Life Safety Performance Level

• N-D: Hazards Reduced Performance Level

In addition, there is the designation of N-E, nonstructural performance not considered, for the situation where only structural improvements are made.

Combining of the performance of both structural and nonstructural components, building performance can be formed.

3-4-1) Structural Performance Levels:

3-4-1-1) Immediate Occupancy Performance Level (S-1):

This performance level is related to the post earthquake damage state in which only very limited structural damage has been appeared. The main vertical and lateral force resisting systems of the building keep nearly all of their pre earthquake strength and stiffness. The risk of life threatening injury because of structural damage is too low, and although some minor structural repairs may be needed, these would generally not be mandatory prior to the reoccupancy.

3-4-1-2) Life Safety Performance Level (S-3):

This performance level is related to the post earthquake damage state in which considerable damage in the structure can be discovered, but some margin against partial or total structural collapse exists. Although some structural elements and components are deeply damaged, but there is not any large falling debris hazards inside or outside of the building. Maybe there exists some injuries during the earthquake but generally the overall risk of life threatening injury as a result of structural damage is low. It will be possible to repair the structure but because of the high expenses this may not be practical.

3-4-1-3) Collapse Prevention Performance Level (S-5):

This performance level is related to the post earthquake damage state in which the building is close to experience partial or total collapse. Considerable damage to the structure has happened, potentially including significant degradation in the stiffness and strength of the lateral force resisting system, permanent lateral deformation of the structure will be large but amount of the decreasing in vertical load carrying capacity will be limited.

However, all of the main elements of the gravity load resisting system must be capable to carry their gravity load demands. High risk of injury because of the falling hazards from structural debris may exist. It is impractical to repair the structure technically and is not safe for reoccupancy, because aftershock activity could cause collapse.

3-4-2) Structural Performance Ranges:

3-4-2-1) Damage Control Performance Range (S-2):

This performance level is related to the post earthquake continuous range of damage states which involve less damage than the Life Safety level, but more than the Immediate Occupancy level. For this range, it is better to design in a way to minimize repair and operation stoppage time as a partial means of protecting valuable equipment and contents or to protect historic features when the cost of design for Immediate Occupancy is high.

By interpolating between the values provided for the Immediate Occupancy (S-1) and Life Safety (S-3) levels, we can find the proper design parameters for this range.

3-4-2-2) Limited Safety Performance Range (S-4):

This performance level is related to the post earthquake continuous range of damage states which involve less damage than the Collapse Prevention level, but more than the Life Safety level. By interpolating between the values provided for the Life Safety (S-3) and Collapse Prevention (S-5) levels, we can find the proper design parameters for this range.

3-4-3) Non Structural Performance Levels:

3-4-3-1) Operational Performance Level (N-A):

This nonstructural performance level is related to the post earthquake damage state in which the nonstructural components are capable to support the building's planned function. At this level, although minor repair of some equipment may be required but most nonstructural systems needed for normal use of the building such as lighting, plumbing, etc. are functional. Also some considerations beyond those that are normally within the sole province of the structural engineer are needed.

3-4-3-2) Immediate Occupancy Performance Level (N-B):

This nonstructural performance level is related to the post earthquake damage state in which only limited nonstructural damage has happened. Basic access and life safety systems, such as doors, stairways, elevators, emergency lighting, fire alarms, and suppression systems, remain functional. Although some of the windows can be broken and maybe some slight damage to some components can be seen too.

Assuming that the building is structurally safe, it is expected that residents could safely remain in the building, although maybe the building cannot be used normally and some cleanup may be needed. Some elements may experience some damage and cannot be functional but generally elements of mechanical and electrical systems in the building are structurally secured and capable to function. Power, water, natural gas, communications lines, and other facilities needed for normal building use may not be reachable. The risk of life threatening injury because of nonstructural damage is very low.

3-4-3-3) Life Safety Performance Level (N-C):

This nonstructural performance level is related to the post earthquake damage state in which potentially deep and costly damage has happened to nonstructural elements but they do not drive out and fall in, threatening life safety inside or outside of the building. Exit routes in the building are not totally blocked. Some injuries may happen during the earthquake because of the nonstructural elements failure but overally the risk of life threatening injury is too low. Damaged nonstructural components can be reconstructed but this may need lots of hard work.

3-4-3-4) Hazards Reduced Performance Level (N-D):

This nonstructural performance level is related to the post earthquake damage state, in which extensive damage has occurred to the nonstructural components but there is not any danger of posing to a hazard for some people because of falling large or heavy items parapets, cladding panels, heavy plaster ceilings, or storage but some serious injury could happen because of falling debris. Failures that could injure large numbers of persons inside or outside the building should be prevented. Emergency exits, fire suppression systems, and similar life safety issues are not addressed in this performance level. **(3- Amr S. Elnashai, Luigi Di Sarno) (6-ATC-1996) (11- FEMA 356)**

3-5) Rehabilitation strategies:

Below some of the most important rehabilitation strategies can be stated:

3-5-1) system completion:

To have a complete lateral force resisting system, the structure should have some basic elements such as diaphragms and walls or frames. The designer can use the system completion strategies to make the system complete or to guarantee that the system behaves as it is designated. In these kinds of structures the capacity spectrum intersects the demand spectra at an acceptable performance point however before reaching that point some local failure events would happen. Common deficiencies that may lead to such kind of local failures include:

-A lack of adequate chord and collector elements at diaphragms

-Inadequate bearing length at precast element supports

-Inadequate anchorage or bracing of structural or nonstructural components. **(6-ATC-1996)**

If the designer tries to add the missing elements to the structure, the structure will be capable to behave in the desired mode. Usually this strategy must be executed with some other strategies to obtain a building with the desired seismic performance characteristics. In the following we will mention the most important elements of this strategy:

3-5-1-1) Chords, Collectors, and Drags:

Diaphragms, chords, collectors, and drags maybe constructed of new reinforced concrete beams/struts or of flush mounted steel plates or members with drilled in anchors. Where there are existing beams, these may be converted to the collector elements by improvement of their capacity or strengthening of the end connections. This method is very common for timber diaphragms or for some older concrete structures.

3-5-1-2) Element Connectivity:

Because most of concrete structures are monolithically constructed, they have adequate nominal interconnection between elements. For the buildings which consist of precast elements, some supplemental interconnection of elements may be required. This can usually be done by adding steel hardware between elements at their end connections.

3-5-1-3) Anchorage and Bracing of Components:

Reaching to some specific performance objectives, the architectural, mechanical, and electrical components of the building must be adequately braced and anchored to resist inertial forces and the drifts that are initiated from the response of the building to the earthquake.

3-5-2) System strengthening and Stiffening:

This strategy is the most common seismic performance rehabilitation approach that can be used for buildings with insufficient lateral force resisting systems. These two mentioned concepts are closely related but different. By strengthening a structure its capacity for bearing of the total lateral force that are required to initiate damage in the structure, increases. If this strengthening is done without stiffening, then the structure can achieve larger lateral displacements without damage but in fact most of the retrofit systems that increase structural strength, such as the addition of walls or frames, will increase structural stiffness too. There are some exceptions such as relatively local retrofit actions that strengthen existing elements without greatly altering their stiffness. For example, in most of the older concrete frames, lap splice lengths are inadequate for longitudinal reinforcing, resulting in a low flexural strength. By providing of the enough confinement around the splices, their performance improves and this allows the frame to develop greater strength without significant impacts on its stiffness. These kinds of measures most probably can improve the frame's deformation capacity too.

In fact system strengthening and stiffening strategies are correlated because most of the systems which strengthen a structure also simultaneously stiffen it too. Similarly stiffening techniques also usually result in a strength increase. Typical systems that are used for stiffening and strengthening include addition of new vertical elements, including shear walls, braced frames, buttresses, or moment resisting frames or diaphragms. In the following we will explain some of the most important elements of this strategy:

3-5-2-1) Shear Walls:

Construction of new shear walls in an existing concrete structure is one of the most common strategies that is used for seismic upgrading. It is an extremely effective method of increasing both building strength and stiffness. Shear walls are compatible with most existing concrete structures and also they are economical.

3-5-2-2) Braced Frames:

Using braced steel frames are another common method of enhancing an existing building's stiffness and strength. Generally braced frames provide lower levels of stiffness and strength but they add less mass to the structure compared to the shear walls. They can be constructed with less disruption of the building and this helps us to loss less light. Also they have a smaller effect on traffic patterns in the building.

3-5-2-3) Buttresses:

Buttresses are braced frames or shear walls. They should be installed perpendicular to an exterior wall of the structure to provide supplemental stiffness and strength. This system is suitable to use specially when during construction the building must remain occupied because most of the construction work, can be done on the building exterior and this minimizes the disturbance to building occupants. Sometimes an additional floor space can be used to buttress the original structure for added seismic resistance.

3-5-2-4) Moment Resisting Frames:

This is an effective strategy to add strength to a building without substantially increasing its stiffness. They can be installed with relatively minimal impact on floor space because they are relatively open.

3-5-2-5) Diaphragm Strengthening:

Usually most of the concrete buildings have adequate diaphragms but there are some exceptions such as presentation of large openings or offsets in the vertical elements of the system which produce locally high demands. Methods of enhancing diaphragms include the provision of topping slabs, metal plates laminated onto the top surface of the slab, or horizontal braced diaphragms beneath the concrete slabs. For buildings with timber diaphragms, diaphragm strengthening can be achieved by increasing the existing nailing in the sheathing, replacing the sheathing with stronger material, or overlaying the existing sheathing with plywood.

3-5-3) Enhancing Deformation capacity:

One of the newer methods to improve the building seismic performance especially for the concrete structures is to increase the ability of the individual elements within the building to resist deformations induced by the building response. Some of the methods in this strategy are adding confinement to the existing elements, making local reductions in stiffness, modifying columns to alter mechanisms, and providing supplemental support at areas subject to deformation induced failure. Compared to the structural strengthening and stiffening, these methods have less effect on the architecture of the building. In the following we will explain some of the most important elements of this strategy:

3-5-3-1) Adding Confinement:

By providing of the exterior confinement jacketing, the deformation capacity of non ductile concrete columns can be improved. Jacketing may consist of continuous steel plates encasing the existing element, reinforced concrete annuluses, and fiber reinforced plastic fabrics.

3-5-3-2) Column Strengthening:

In most of the old concrete structures, beams are stronger than columns. These kinds of structures tend to have single story mechanisms and all of the inelastic deformation demand initiated because of earthquake develops in the story in which the mechanism has created. Because in such cases displacement demands is concentrated in a single story, the columns can experience very large local inelastic deformation demands at relatively low levels of total structure lateral displacement. So it is better to strengthen the columns such that the beams become the weaker elements, this will allow us to have much larger overall structural drifts to be reached and will prevent formation of story mechanisms.

3-5-3-3) Local Stiffness Reductions:

To prevent unwanted damage modes, local reductions in stiffness can be a useful way. Also this strategy is useful to minimize damage to a few scattered elements that their actions are not critical for the building's overall performance. Because in many of older concrete structures some deep spandrels exist, there is a danger to have some short column failures at perimeter walls. Probability of having this kind of failure can be reduced by establishing joints between the face of the column and adjacent architectural elements, such as spandrel panels or infills that create the condition. Some buildings may have one or more walls that are constructed for architectural reasons and they do not have a critical structural action. These walls may be too stiff and sometimes they can attract more lateral force than they can resist. Even they can have some torsional response or they can introduce discontinuous load paths into the structure. By Local demolition of these elements or modification of them we can reduce their stiffness and this can result in a cost effective performance improvement for the structure.

3-5-3-4) Supplemental Support:

There are some deficient gravity load bearing elements which do not do a critical action in the lateral force resisting system of the structure but whose support can be endangered by large lateral building deflections. This method can be useful to improve their action. For example, flat slabs may be subjected to punching shear failures due to lateral building deformations. To improve their behavior, some supplemental bearing supports at columns could be provided. As another example, we can provide some supplemental bearing supports for precast beams which have inadequate bearing length.

3-5-4) Reducing Earthquake Demands:

Sometimes instead of increasing of the capacity of the building to resist earthquake induced forces and deformations, it is better to decrease the demand forces and deformations by some kind of modification. In fact, the demand spectrum for the structure, rather than the capacity spectrum, is modified. Methods for this strategy are such as reductions in the building's mass and the installation of systems for base isolation and/or energy dissipation. Unfortunately the mentioned instruments are more expensive compared to the other rehabilitation methods so these kinds of methods are usually proper for the buildings that are more important and have critical occupancies with sensitive equipment or it is needed to function rapidly after the earthquake. These kinds of strategies are also more attractive for the retrofitting of the historic structures because for applying such kind of strategies we need less extensive invasive construction. It should be mentioned that the designer should not rely a lot to these kinds of strategies for retrofitting of the historic buildings because in fact many structures employing a strategy of reducing earthquake demands also require some additional strengthening and stiffening. In the following we will explain some of the most important elements of this strategy:

3-5-4-1) Base Isolation:

In this method some compliant bearings within a single level of the building's vertical load carrying system will be installed. It is better to install them near the base. The bearings are designed to have relatively low lateral stiffness, extensive lateral deformation capacity and may also have better energy dissipation characteristics. By installation of an isolation system, a considerable increase in the building's fundamental response period and its effective damping can be seen. Since the isolation bearings have much greater lateral compliance than does the structure itself. Lateral deformation demands produced by the earthquake tend to concentrate in the bearings themselves. All of the above mentioned hints help the structure that is located above the isolation bearings to greatly reduce the lateral demands.

The amount of energy dissipation in a given displacement for a structure that is base isolated is clearly different from that which occurs for the same structure with a fixed base condition. So the designer should recompute the effective damping of the family of demand curves at various maximum structural displacements to be able to overlay the demand and the capacity spectra for a base isolated structure. Also it should be mentioned that by installation of the base isolation bearings significant reductions in overall structural stiffness will be took place and so it takes greater displacements to reach to the same effective damping. However, because the isolated system is capable of safety accommodating bigger displacements than the fixed base structure, it will be also capable of mobilizing much larger effective damping when it is isolated.

3-5-4-2) Energy Dissipation Systems:

By applying this method the ability of the structure to dampen earthquake response with a benign behavior through either viscous or hysteretic damping will increases. This method requires the installation of energy dissipation units (EDUs) within the lateral force resisting system. The EDUs dissipate energy and in this process reduce the displacement demands on the structure. Before installation of EDUs usually we need to install some vertical braced frames to act as a mounting platform for the units and usually this lead to a simultaneous increase in system stiffness too. Generally energy dissipation systems are more expensive than the common systems that are used for stiffening and strengthening of a building but they have the potential to improve the performance. The most important effect however is on the demand spectra. The efficiency of the EDUs in dissipating energy leads to a much greater effective damping at any displacement.

3-5-4-3) Mass Reduction:

We can greatly improve the performance of some buildings by reducing the building mass. By reducing of the building mass, the building's natural period, the amount of inertial force that develops during its response and the total displacement demand on the structure will reduce too. This strategy can be applied by elimination of the heavy nonstructural elements, such as cladding, water tanks, and storage. In the critical situations, mass reduction can be done by removing one or more building stories. **(6-ATC-1996) (11- FEMA 356)**

3-6) Push-over Analysis in SAP 2000:

In SAP2000, a frame element is modeled as a line element having linearly elastic properties and nonlinear force-displacement characteristics and then the hinge properties for different frames (beams and columns) can be defined and assigned to all of them. The force-deformation diagram for an element is shown in below:

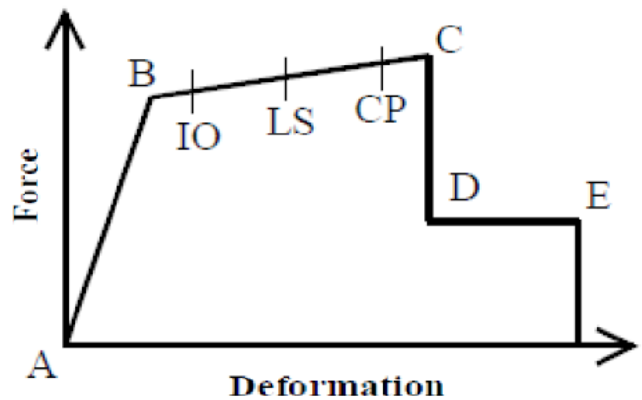

Figure 3-8- Force-deformation diagram

Point A is related to the unloaded condition and point B represents yielding of the element. The ordinate at C is related to nominal strength and abscissa at C is related to the deformation at which significant strength degradation begins. The sudden fall from C to D represents the initial failure of the element and resistance to lateral loads beyond point C is usually unreliable. The remaining resistance from D to E allows the frame elements to bear the gravity loads. Beyond point E, the maximum deformation capacity, gravity load cannot be sustained anymore. Hinges can be assigned at any number of locations (potential yielding points) along the span of the frame element as well as element ends. Uncoupled moment (M2 andM3), torsion (T), axial force (P) and shear (V2 and V3) force displacement relations can be defined. When the axial load of the column changes under lateral loading, there is also a coupledP-M2-M3 (PMM) hinge which yields based on the interaction of axial force and bending moments at the hinge location. Also, 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: **default hinge properties, user-defined hinge properties and generated hinge properties.** We only can assign the “default hinge properties” and “user-defined hinge properties” to the frame elements. **(5- Ashraf Habibullah and Stephen Pyle) (8-Chung- Yue Wang and Shaing-Yung Ho) (22-Sermin Oguz)**

4) Non-linear Static Push-over Analysis in Eurocode(Eurocode 8-Part 3):

4-1) Performance Requirements and compliance Criteria:

4-1-1) Fundamental requirements:

Based on the Eurocode three Limit States (LS), which is called Near Collapse (NC), Significant Damage (SD), and Damage Limitation (DL) can be defined as follows:

LS of Near Collapse (NC). The structure is heavily damaged, with a limited remaining lateral strength and stiffness, although vertical elements are still capable of bearing vertical loads. Most of the non structural components have collapsed. Large permanent drifts are present. The structure is near collapse and would probably not sustain another earthquake, even with a moderate intensity.

LS of Significant Damage (SD). The structure is heavily damaged, with a limited remaining lateral strength and stiffness, although vertical elements are still capable of bearing vertical loads. Non structural components are damaged, although partitions and infills have not failed out of plane. Moderate permanent drifts are present. The structure can withstand after shocks with moderate intensity but repairing of the structure will be uneconomic.

LS of Damage Limitation (DL). The structure is only slightly damaged, most of the structural elements are prevented from significant yielding and they keep their strength and stiffness properties. Non structural components, such as partitions and infills, may crack widely, but they could be repaired economically. Permanent drifts are not significant and the structure does not need any repair measures.

For each limit state a return period for the seismic action should be selected and this can be done by help of the National Annex of each country but also the below values can be used for new buildings:

– LS of Near Collapse (NC): 2.475 years, corresponding to a probability of exceedance of 2% in 50 years

– LS of Significant Damage (SD): 475 years, corresponding to a probability of exceedance of 10% in 50 years

– LS of Damage Limitation (DL): 225 years, corresponding to a probability of exceedance of 20% in 50 years.

4-1-2) Compliance criteria:

Before evaluation of the structural elements, they should be grouped into two main groups: 'ductile' and 'brittle'. Except when using the q-factor approach, the former shall be verified by checking that demands do not exceed the corresponding capacities regarding to the deformations. The latter shall be verified by checking that demands do not exceed the corresponding capacities in terms of strengths. Also there are some other structural elements that are called "secondary seismic". These elements shall be checked with the same criteria as primary seismic ones, but just we should use less conservative estimates of their capacity.

Also a q factor method may be used, and in that case all of structural elements shall be verified by checking that demands due to the reduced seismic action do not exceed the corresponding capacities in terms of strengths.

For the calculation of the capacities of ductile or brittle elements, average value properties of the existing materials shall be used. These values can directly be obtained from *in-situ* tests and from the additional sources of information and then they should be divided by the confidence.

In the near collapse (NC) limit state, for ductile elements ultimate deformations and for the brittle ones ultimate strengths should be considered in calculation of the capacities. The q-factor is generally not applicable for checking this Limit State.

The values of q = 1.5 and 2.0 can be recommended to be used for reinforced concrete and steel structures respectively, also the higher values of q can be used based on the local and global available ductility and the definition of the Significant Damage Limit State.

In the significant damage limit state, capacities shall be based on damage related deformations for ductile elements and on conservatively estimated strengths for brittle ones. Except when using the q-factor method, demands shall be based on the reduced seismic action and capacities shall be evaluated same as for non seismic design situations.

In the damage limitation (DL) limit state, capacities shall be based on yield strengths for both of the ductile and brittle structural elements. Except when using the q-factor method, comparison between the demands and capacities should be done based on the average inter storey drift.

4-1-3) Knowledge levels:

There are three knowledge levels that can be used for the selection of the system of analysis; Limited knowledge (KL1), Normal knowledge (KL2), Full knowledge (KL3).

There are some factors that can be used for selection of the knowledge level such as geometrical properties of the structural system, detail of the reinforcements and the connections, and the mechanical properties of the materials.

In the Limited knowledge level, the geometry of the structure and member sizes are known either from construction drawings or from observation. From detailed construction drawings the structural details are not known and they can be assumed based on the simulated design in accordance with usual practice at the time of construction and no direct information from the mechanical properties of the used materials is available, either from original design information or from original test reports. Default values should be assumed in accordance with standards at the time of construction. Structural evaluation based on a state of limited knowledge should be done with linear analysis methods, either static or dynamic.

In the normal knowledge level, the geometry of the structure and member sizes are known either from outline construction drawings or from a survey. The structural details are known either from incomplete detailed construction drawings or from extended *in-situ* inspection. The mechanical properties of the used materials is available either from original design specifications or from extended *in-situ* tests. Structural evaluation based on this state of knowledge may be done through either linear or nonlinear analysis methods, either static or dynamic.

In the full knowledge level, the geometry of the structure and member sizes are known overally either from the complete set of construction drawings or from an extensive survey. The structural details are known either from a complete set of detailed construction drawings or from extensive *in-situ* inspection. The mechanical properties of the used materials are available either from original test reports or from extensive *in-situ* tests.

In the below table the relationship between knowledge levels and applicable methods of analysis and confidence factors is illustrated:

Knowledge Level	Geometry	Details	Materials	Analysis	CF
KL1	From original outline construction drawings with sample **visual** survey ***or*** from **full** survey	Simulated design in accordance with relevant practice ***and*** from **limited** *in-situ* inspection	Default values in accordance with standards of the time of construction ***and*** from **limited** *in-situ* testing	LF- MRS	CF_{KL1}
KL2		From incomplete original detailed construction drawings with **limited** *in-situ* inspection ***or*** from **extended** *in-situ* inspection	From original design specifications with **limited** *in-situ* testing ***or*** from **extended** *in-situ* testing	All	CF_{KL2}
KL3		From original detailed construction drawings with **limited** *in-situ* inspection ***or*** from **comprehensive** *in-situ* inspection	From original test reports with **limited** *in-situ* testing ***or*** from **comprehensive** *in-situ* testing	All	CF_{KL3}

NOTE The values ascribed to the confidence factors to be used in a country may be found in its National Annex. The recommended values are CF_{KL1} = 1,35, CF_{KL2} = 1,20 and CF_{KL3} = 1,00.

Table 4-1- Relationship between knowledge levels -Methods of analysis -Confidence factors

4-2) Structural Assessment:

To find out that an existing undamaged or damaged building can fulfill the required limit state appropriate to the seismic action or not, an assessment should be done. For the analysis one of the below methods can be used:

– lateral force analysis (linear),

– modal response spectrum analysis (linear),

– non-linear static (pushover) analysis,

– non-linear time history dynamic analysis.

– *q*-factor approach.

In the following the nonlinear static analysis (pushover analysis) will be described.

4-2-1) Nonlinear static analysis (pushover analysis):

In the nonlinear static analysis the structure will be subjected to the constant gravity loads and monotonically increasing horizontal loads. In this procedure at least two vertical distributions of lateral loads should be applied: “uniform” and “modal” patterns. First pattern is based on lateral forces that are proportional to mass regardless of height (uniform response acceleration) and the second one is proportional to lateral forces consistent with the lateral force distribution which is determined in the elastic analysis. Lateral loads should be applied at the location of the masses in the model. Then the relation between base shear force and the control displacement should be extracted based on a target displacement and then the capacity curve graph can be plotted.

4-3) Interventions:

4-3-1) Type of intervention:

The intervention strategies should be able to improve the capacity of lateral force resisting systems and horizontal diaphragms and decrease the demand imposed by seismic actions. The retrofitted structural systems should have homogeneous mass, stiffness and strength (the distribution of them). The latter issue help the structure not to have torsional effects and soft storey mechanisms. Also they should have enough mass and stiffness to avoid highly flexible structures. Finally they should guarantee to have a uniform load path and prevent brittle failures. There are different types of intervention that can be used some of them are summarizes below:

a) Local or overall modification of damaged or undamaged elements (repair, strengthening or full replacement).

b) Addition of new structural elements (e.g. bracings or infill walls; steel, etc);

c) Modification of the structural system (widening of joints, elimination of some structural joints, elimination of vulnerable elements, modification into more regular or more ductile arrangements);

d) Addition of a new structural system to bear some or all of the entire seismic action;

e) Transformation of existing non structural members into structural members;

f) Installation of passive protection devices through either base isolation or dissipative bracing

g) Mass reduction (Removal of unused equipment and storage loads, Replacement of heavy cladding systems with lighter systems, replacement of masonry partition walls with lighter systems, removal of one or more storeys,…)

h) Restriction or changing of building application;

i) Partial demolition;

j) Enhancement of ductility of the structure.

k) Seismic isolation.

l) Supplemental damping.

Also more than one of these strategies can be selected at the same time for a structure.

4-3-2) Non-structural elements:

There are different types of interventions that can be used. Repairing or strengthening of non structural elements is sometimes important because in addition to functional requirements, the seismic behavior of these kinds of elements may endanger the life of occupants. Some of them are summarizes below:

a) Appropriate connections to structural elements.

b) Increasing the resistance of non structural elements.

c) Taking measures of anchorage to prevent possible falling out of parts of these elements.

4-3-3) Strengthening of the concrete structures:

4-3-3-1) Concrete jacketing:

Applying the concrete jackets for columns and walls can increase the bearing capacity, flexural and shear strength, deformation capacity or strength of deficient lap splices. The thickness of the jackets should be such that longitudinal and transverse reinforcement can be placed with an adequate cover. If increasing of the flexural strength is needed, longitudinal bars should be continued to the related storey through holes in the slab, while horizontal ties should be placed in the joint region through horizontal holes drilled in the beams.

4-3-3-2) Steel jacketing:

Applying the steel jackets for columns can increase the shear strength and improve the strength of deficient lap splices. They can also increase ductility through confinement. They are usually consist of four corner angles to which either continuous steel plates, or thicker discrete horizontal steel straps, are welded. Corner angles may be epoxy bonded to the concrete or just adhere to it without gaps along the entire height. Straps may be pre heated just before the welding, in order to provide some positive confinement on the column.

4-3-3-3) FRP plating and wrapping:

Applying of the FRP (fiber reinforced polymers) can improve the shear capacity of columns and walls and the ductility of the members at their ends and also can prevent the lap splice failure, with the help of added confinement in the form of FRP jackets, with the fibers oriented along the perimeter.

4-3-4) Strengthening of the steel structures:

4-3-4-1) Connection retrofitting (Beam-to-column connections):

The most important goal is to remove the beam plastic hinge away from the column face. This kind of retrofitting can be done with weld replacement, or a weakening strategy, or a strengthening strategy.

4-3-4-1-1) Weld replacement:

The old filler material should be removed and replaced with a new material with high quality. For strengthening and stiffening of the column panel transverse stiffeners at the top and bottom of the panel zone should be used. Transverse and web stiffeners should be welded to column flanges and to the web with complete joint penetration welds.

4-3-4-1-2) Weakening strategies:

A) Connections with RBS beams:

By Reduced Beam Sections (RBS), formation of the plastic hinges will be transferred to the reduced section and this can decrease the probability of the fracture at the beam flange welds and in the nearby heat affected zones. The beam should be connected to the column flange either with welded webs or with shear tabs welded to the column flange face and to the beam web.

B) Semi-rigid connections:

If the goal is to reach to the large plastic deformations without risk of fracture, then applying of the semi rigid or partial strength connections (steel or composite) will be useful. For design, it can be assumed that the shear resistance is provided by the components on the web and the flexural resistance by the beam flanges and the slab reinforcement, if any.

4-3-4-1-3) Strengthening strategies:

A) Haunched connections:

By adding haunches, beam to column connections may be strengthened. These haunches can only be added to the bottom, or to the top and the bottom of the beam flanges. Although adding of the haunches to the bottom flange is easier, because bottom flanges are more accessible than top ones and also the composite slab (if it exists), does not have to be removed. The most effective type of haunch is triangular T-shaped one. Transverse stiffeners at the level of the top and bottom beam flanges and also at the haunch edges should be used to strengthen the column panel zone and the column web and the beam web respectively. They should be completely welded to the both column and the beam flanges.

B) Cover plate connections:

These kinds of connections can reduce the stress at the welds of the beam flange and they can force yielding in the beam to occur at the end of the cover plates. They can be used only at the bottom beam flange, or at the top and bottom beam flanges at the same time. **(10-Eurocode 8- Part 3)**

5) Geometry:

In the following chapters the inelastic methods presented above, will be applied for the analysis of a reinforced concrete building. The results will be presented and finally an appropriate type of intervention will be considered to improve the seismic behavior of the building. This residential building has seven stories. The first four stories are bigger while the three remaining stories are in a setback. The characteristics of the stories can be summarized as below:

5-1) Frame Sections:

The sections that are used for beams and columns can be summarized as follows:

5-1-1) Columns:

The cross sections of the columns will be shown in below:

A) Underground to 5th floor:

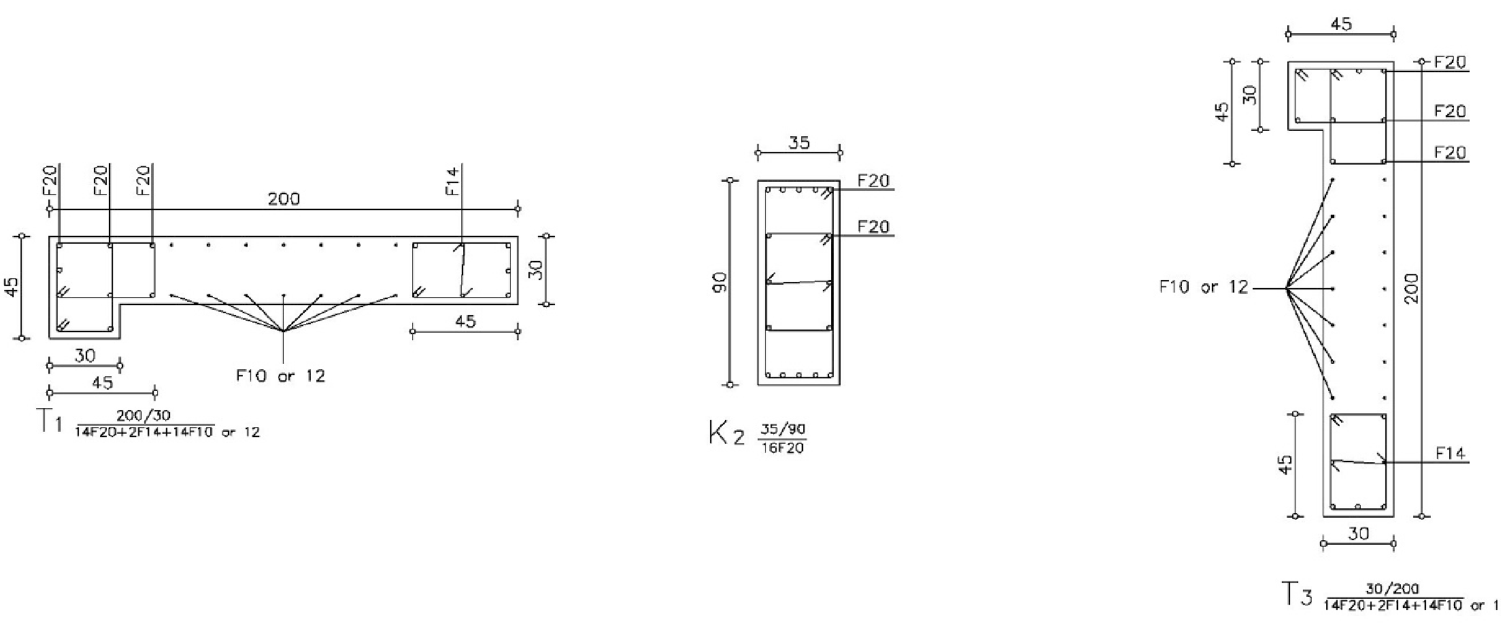

Figure 5-1- Cross sections of the columns-Part 1

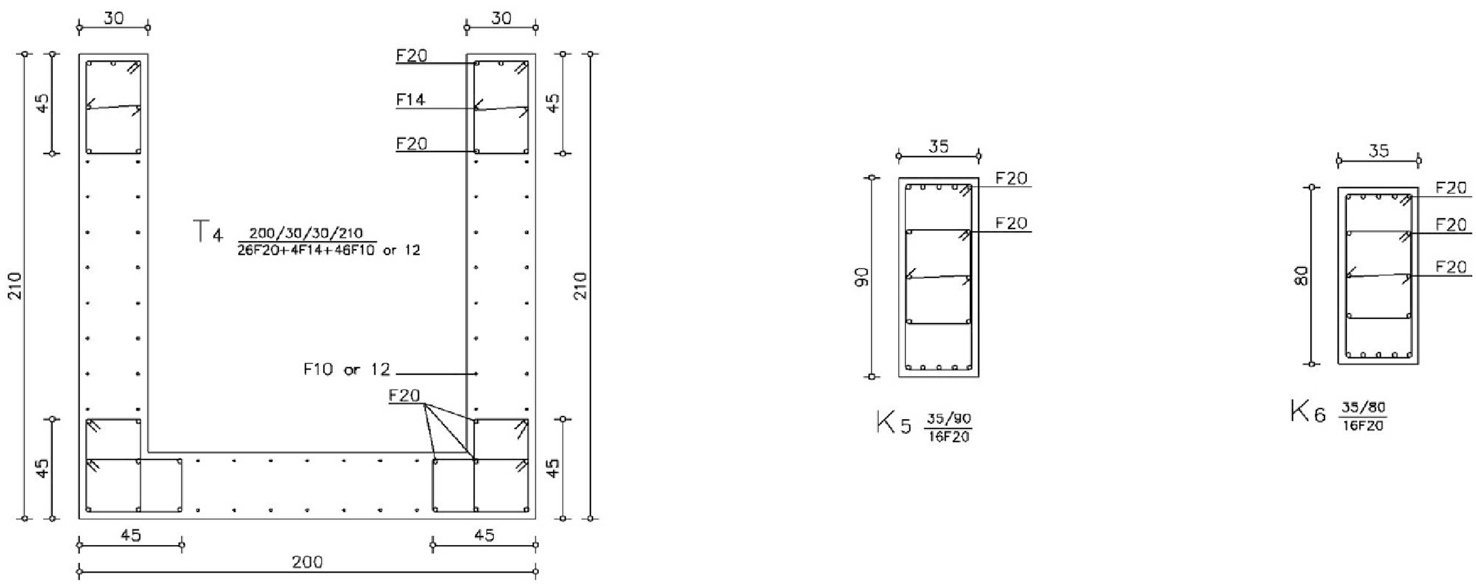

Figure 5-2- Cross sections of the columns-Part 2

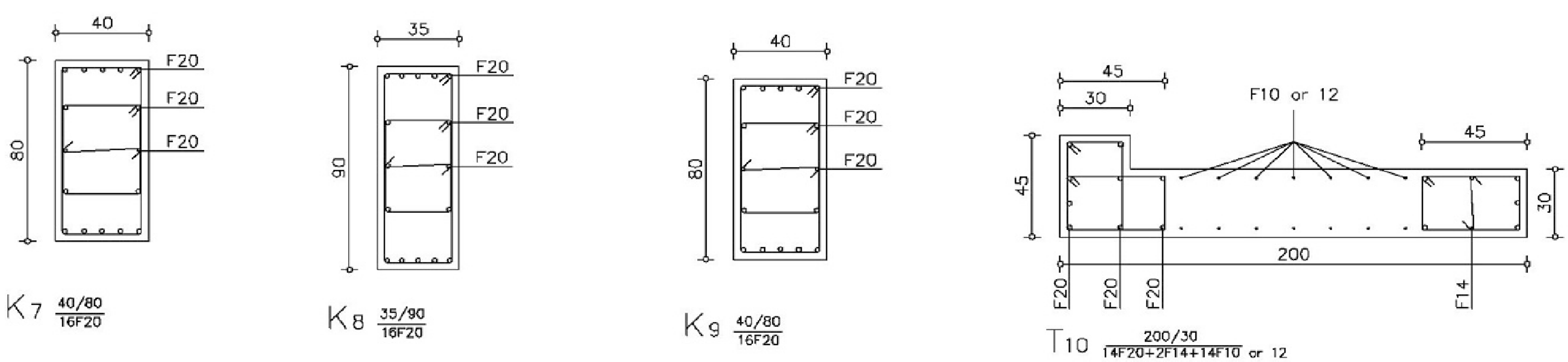

Figure 5-3- Cross sections of the columns-Part 3

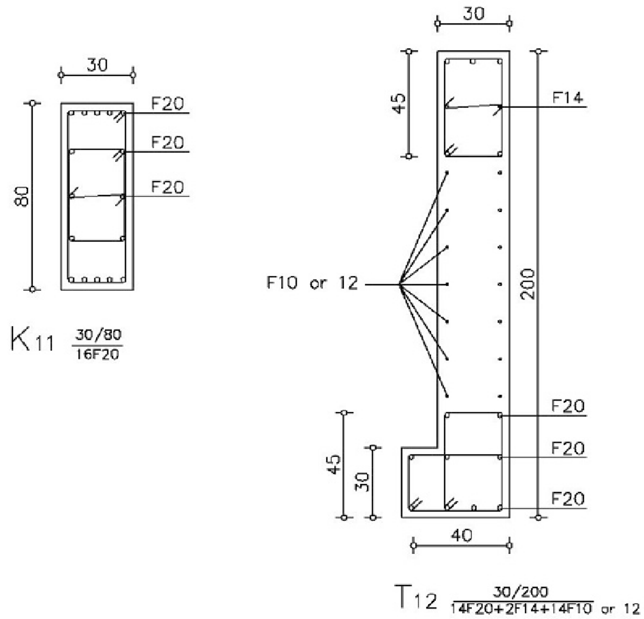

Figure 5-4- Cross sections of the columns-Part 4

B) Last storey (6th or Lift room):

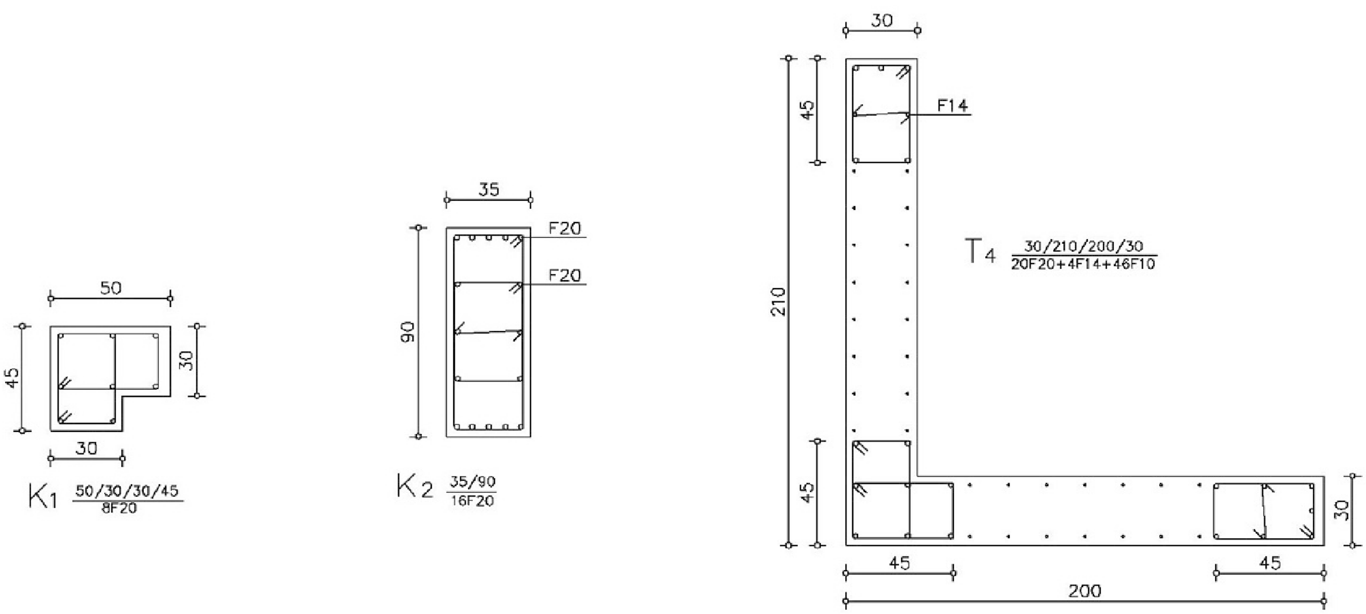

Figure 5-5- Cross sections of the columns-Part 5

The detailed cross section of the columns in each storey with their related reinforcement can be summarized in the tables below:

	T1			K2		
	Section(cm×cm)	Reinforcement		Section(cm×cm)	Reinforcement	
		Rebars	Area(m²)		Rebars	Area(m²)
Under Ground	200×30	14 Φ 20 + 2 Φ 14 + 14 Φ 12	6.29E-03	35×90	16 Φ 20	5.03E-03
Ground Floor	200×30	14 Φ 20 + 2 Φ 14 + 14 Φ 12	6.29E-03	35×90	16 Φ 20	5.03E-03
1st	200×30	14 Φ 20 + 2 Φ 14 + 14 Φ 12	6.29E-03	35×90	16 Φ 20	5.03E-03
2nd	200×30	14 Φ 20 + 2 Φ 14 + 14 Φ 12	6.29E-03	35×90	16 Φ 20	5.03E-03
3rd	200×30	14 Φ 20 + 2 Φ 14 + 14 Φ 10	5.81E-03	35×90	16 Φ 20	5.03E-03
4th	200×30	14 Φ 20 + 2 Φ 14 + 14 Φ 10	5.81E-03	35×90	16 Φ 20	5.03E-03
5th	200×30	14 Φ 20 + 2 Φ 14 + 14 Φ 10	5.81E-03	35×90	16 Φ 20	5.03E-03
(6th) Lift Room	50×30/30×45	8 Φ 20	2.51E-03	35×90	16 Φ 20	5.03E-03

Table 5-1- Detailed Cross sections of the columns-Part 1

	T3			T4		
	Section(cm×cm)	Reinforcement		Section(cm×cm)	Reinforcement	
		Rebars	Area(m²)		Rebars	Area(m²)
Under Ground	30×200	14 Φ 20 + 2 Φ 14 + 14 Φ 12	6.29E-03	200×30/30×210	26 Φ 20 + 4 Φ 14 + 46 Φ 12	1.40E-02
Ground Floor	30×200	14 Φ 20 + 2 Φ 14 + 14 Φ 12	6.29E-03	200×30/30×210	26 Φ 20 + 4 Φ 14 + 46 Φ 12	1.40E-02
1st	30×200	14 Φ 20 + 2 Φ 14 + 14 Φ 12	6.29E-03	200×30/30×210	26 Φ 20 + 4 Φ 14 + 46 Φ 12	1.40E-02
2nd	30×200	14 Φ 20 + 2 Φ 14 + 14 Φ 12	6.29E-03	200×30/30×210	26 Φ 20 + 4 Φ 14 + 46 Φ 12	1.40E-02
3rd	30×200	14 Φ 20 + 2 Φ 14 + 14 Φ 10	5.81E-03	200×30/30×210	26 Φ 20 + 4 Φ 14 + 46 Φ 12	1.40E-02
4th	30×200	14 Φ 20 + 2 Φ 14 + 14 Φ 10	5.81E-03	200×30/30×210	26 Φ 20 + 4 Φ 14 + 46 Φ 10	1.24E-02
5th	30×200	14 Φ 20 + 2 Φ 14 + 14 Φ 10	5.81E-03	200×30/30×210	26 Φ 20 + 4 Φ 14 + 46 Φ 10	1.24E-02
(6th) Lift Room				30×210/200×30	20 Φ 20 + 4 Φ 14 + 46 Φ 10	1.05E-02

Table 5-2- Detailed Cross sections of the columns-Part 2

	K5			K6			K7		
	Section(cm×cm)	Reinforcement		Section(cm×cm)	Reinforcement		Section(cm×cm)	Reinforcement	
		Rebars	Area(m²)		Rebars	Area(m²)		Rebars	Area(m²)
Under Ground	90×35	16 Φ 20	5.03E-03	80×35	16 Φ 20	5.03E-03	80×40	16 Φ 20	5.03E-03
Ground Floor	90×35	16 Φ 20	5.03E-03	80×35	16 Φ 20	5.03E-03	80×40	16 Φ 20	5.03E-03
1st	90×35	16 Φ 20	5.03E-03	80×35	16 Φ 20	5.03E-03	80×40	16 Φ 20	5.03E-03
2nd	90×35	16 Φ 20	5.03E-03	80×35	16 Φ 20	5.03E-03	80×40	16 Φ 20	5.03E-03
3rd	90×35	16 Φ 20	5.03E-03	80×35	16 Φ 20	5.03E-03	80×40	16 Φ 20	5.03E-03
4th	90×35	16 Φ 20	5.03E-03	80×35	16 Φ 20	5.03E-03	80×40	16 Φ 20	5.03E-03
5th	90×35	16 Φ 20	5.03E-03	80×35	16 Φ 20	5.03E-03	80×40	16 Φ 20	5.03E-03
(6th) Lift Room									

Table 5-3- Detailed Cross sections of the columns-Part 3

	K8			K9			T10		
	Section(cm×cm)	Reinforcement		Section(cm×cm)	Reinforcement		Section(cm×cm)	Reinforcement	
		Rebars	Area(m²)		Rebars	Area(m²)		Rebars	Area(m²)
Under Ground	90×35	16 Φ 20	5.03E-03	80×40	16 Φ 20	5.03E-03	200×30	14 Φ 20 + 2 Φ 14 + 14 Φ 12	6.29E-03
Ground Floor	90×35	16 Φ 20	5.03E-03	80×40	16 Φ 20	5.03E-03	200×30	14 Φ 20 + 2 Φ 14 + 14 Φ 12	6.29E-03
1st	90×35	16 Φ 20	5.03E-03	80×40	16 Φ 20	5.03E-03	200×30	14 Φ 20 + 2 Φ 14 + 14 Φ 12	6.29E-03
2nd	90×35	16 Φ 20	5.03E-03	80×40	16 Φ 20	5.03E-03	200×30	14 Φ 20 + 2 Φ 14 + 14 Φ 12	6.29E-03
3rd	90×35	16 Φ 20	5.03E-03	80×40	16 Φ 20	5.03E-03			
4th	90×35	16 Φ 20	5.03E-03	80×40	16 Φ 20	5.03E-03			
5th	90×35	16 Φ 20	5.03E-03	80×40	16 Φ 20	5.03E-03			
(6th) Lift Room									

Table 5-4- Detailed Cross sections of the columns-Part 4

	K11			T12		
	Section(cm×cm)	Reinforcement		Section(cm×cm)	Reinforcement	
		Rebars	Area(m²)		Rebars	Area(m²)
Under Ground	30×80	16 Φ 20	5.03E-03	30×200	14 Φ 20 + 2 Φ 14 + 14 Φ 12	6.29E-03
Ground Floor	30×80	16 Φ 20	5.03E-03	30×200	14 Φ 20 + 2 Φ 14 + 14 Φ 12	6.29E-03
1st	30×80	16 Φ 20	5.03E-03	30×200	14 Φ 20 + 2 Φ 14 + 14 Φ 12	6.29E-03
2nd	30×80	16 Φ 20	5.03E-03	30×200	14 Φ 20 + 2 Φ 14 + 14 Φ 12	6.29E-03
3rd						
4th						
5th						
(6th) Lift Room						

Table 5-5- Detailed Cross sections of the columns-Part 5

5-1-2) Beams:

The detailed cross section of the Beams in each storey with their related reinforcement can be summarized in the following tables:

A) Ground Floor:

Ground Floor(z=+0.00m)

Beam	Section (cm×cm)	Reinforcement			
		Top		Bottom	
		Rebars	Area(m^2)	Rebars	Area(m^2)
1	30×50	3 Φ 20	9.42E-04	3 Φ 20	9.42E-04
2	30×50	2 Φ 20	6.28E-04	3 Φ 20	9.42E-04
3	30×50	3 Φ 20	9.42E-04	3 Φ 20	9.42E-04
4	30×50	2 Φ 18	5.09E-04	3 Φ 18	7.63E-04
5	30×50	3 Φ 20	9.42E-04	4 Φ 20	1.26E-03
6	30×50	2 Φ 20	6.28E-04	3 Φ 20	9.42E-04
7	30×50	3 Φ 16	6.03E-04	3 Φ 16	6.03E-04
8	30×50	3 Φ 20	9.42E-04	3 Φ 20	9.42E-04
9	40×50	3 Φ 18	7.63E-04	4 Φ 18	1.02E-03
10	30×50	3 Φ 20	9.42E-04	3 Φ 20	9.42E-04
11	30×50	3 Φ 20	9.42E-04	3 Φ 20	9.42E-04
12	40×50	3 Φ 18	7.63E-04	3 Φ 18	7.63E-04
13	30×50	3 Φ 20	9.42E-04	3 Φ 20	9.42E-04
14	30×50	3 Φ 18	7.63E-04	3 Φ 18	7.63E-04
15	30×50	3 Φ 20	9.42E-04	3 Φ 20	9.42E-04
16	30×50	3 Φ 20	9.42E-04	3 Φ 20	9.42E-04
17	30×50	3 Φ 18	7.63E-04	3 Φ 18	7.63E-04
18	30×50	3 Φ 20	9.42E-04	3 Φ 20	9.42E-04

Table 5-6- Detailed Cross sections of the Beams-Part 1

B) First Floor:

First Floor(z=+2.80m)

Beam	Section (cm×cm)	Reinforcement			
		Top		Bottom	
		Rebars	Area(m^2)	Rebars	Area(m^2)
1	30×50	3 Φ 20	9.42E-04	3 Φ 20	9.42E-04
2	30×50	2 Φ 20	6.28E-04	3 Φ 20	9.42E-04
3	30×50	3 Φ 20	9.42E-04	3 Φ 20	9.42E-04
4	30×50	3 Φ 18	7.63E-04	3 Φ 18	7.63E-04
5	30×50	3 Φ 20	9.42E-04	4 Φ 20	1.26E-03
6	30×50	2 Φ 20	6.28E-04	3 Φ 20	9.42E-04
7	30×50	3 Φ 16	6.03E-04	3 Φ 16	6.03E-04
8	30×50	3 Φ 20	9.42E-04	3 Φ 20	9.42E-04
9	40×50	2 Φ 18	5.09E-04	4 Φ 18	1.02E-03
10	30×50	3 Φ 20	9.42E-04	3 Φ 20	9.42E-04
11	30×50	3 Φ 20	9.42E-04	3 Φ 20	9.42E-04
12	40×50	2 Φ 18	5.09E-04	4 Φ 18	1.02E-03
13	30×50	3 Φ 20	9.42E-04	3 Φ 20	9.42E-04
14	30×50	3 Φ 18	7.63E-04	3 Φ 18	7.63E-04
15	30×50	3 Φ 20	9.42E-04	3 Φ 20	9.42E-04
16	30×50	3 Φ 20	9.42E-04	3 Φ 20	9.42E-04
17	30×50	3 Φ 18	7.63E-04	3 Φ 18	7.63E-04
18	30×50	3 Φ 20	9.42E-04	3 Φ 20	9.42E-04

Table 5-7- Detailed Cross sections of the Beams-Part 2

C) Second & Third Floors:

Second & Third Floors(z=+5.80m,z=+8.80m)

Beam	Section (cm×cm)	Reinforcement			
		Top		Bottom	
		Rebars	Area(m^2)	Rebars	Area(m^2)
1	30×60	3 Φ 20	9.42E-04	3 Φ 20	9.42E-04
2	30×60	2 Φ 20	6.28E-04	3 Φ 20	9.42E-04
3	30×60	3 Φ 20	9.42E-04	3 Φ 20	9.42E-04
4	30×60	3 Φ 18	7.63E-04	3 Φ 18	7.63E-04
5	30×60	3 Φ 20	9.42E-04	4 Φ 20	1.26E-03
6	30×60	2 Φ 20	6.28E-04	3 Φ 20	9.42E-04
7	30×60	3 Φ 16	6.03E-04	3 Φ 16	6.03E-04
8	30×60	3 Φ 20	9.42E-04	3 Φ 20	9.42E-04
9	60×40	4 Φ 18	1.02E-03	6 Φ 18	1.53E-03
10	30×60	3 Φ 20	9.42E-04	3 Φ 20	9.42E-04
11	30×60	3 Φ 20	9.42E-04	3 Φ 20	9.42E-04
12	60×40	4 Φ 18	1.02E-03	4 Φ 18	1.02E-03
13	30×60	3 Φ 20	9.42E-04	3 Φ 20	9.42E-04
14	30×60	3 Φ 20	9.42E-04	3 Φ 20	9.42E-04
15	30×60	3 Φ 20	9.42E-04	3 Φ 20	9.42E-04
16	30×60	3 Φ 20	9.42E-04	3 Φ 20	9.42E-04
17	30×60	3 Φ 20	9.42E-04	3 Φ 20	9.42E-04
18	30×60	3 Φ 20	9.42E-04	3 Φ 20	9.42E-04

Table 5-8- Detailed Cross sections of the Beams-Part 3

D) Fourth & Fifth Floors:

Fourth & Fivth Floors(z=+11.80m,z=+14.80m)

Beam	Section (cm×cm)	Reinforcement			
		Top		Bottom	
		Rebars	Area(m^2)	Rebars	Area(m^2)
1	30×60	3 Φ 20	9.42E-04	3 Φ 20	9.42E-04
2	30×60	2 Φ 20	6.28E-04	3 Φ 20	9.42E-04
3	30×60	3 Φ 20	9.42E-04	3 Φ 20	9.42E-04
4	30×60	3 Φ 18	7.63E-04	3 Φ 18	7.63E-04
5	30×60	3 Φ 20	9.42E-04	4 Φ 20	1.26E-03
6	30×60	2 Φ 20	6.28E-04	3 Φ 20	9.42E-04
7	30×60	3 Φ 16	6.03E-04	3 Φ 16	6.03E-04
8	30×60	3 Φ 20	9.42E-04	3 Φ 20	9.42E-04
9	40×60	3 Φ 18	7.63E-04	5 Φ 18	1.27E-03
10	30×60	3 Φ 20	9.42E-04	3 Φ 20	9.42E-04
11	30×60	3 Φ 20	9.42E-04	3 Φ 20	9.42E-04
12	40×60	2 Φ 18	5.09E-04	3 Φ 18	7.63E-04
13	30×60	3 Φ 20	9.42E-04	3 Φ 20	9.42E-04

Table 5-9- Detailed Cross sections of the Beams-Part 4

E) Sixth & Seventh Floors:

Sixth Floor(z=+17.80m)

Beam	Section (cm×cm)	Reinforcement			
		Top		Bottom	
		Rebars	Area(m^2)	Rebars	Area(m^2)
1	30×60	3 Φ 20	9.42E-04	3 Φ 20	9.42E-04
2	30×60	2 Φ 20	6.28E-04	3 Φ 20	9.42E-04
3	30×60	3 Φ 20	9.42E-04	3 Φ 20	9.42E-04
4	30×60	3 Φ 18	7.63E-04	3 Φ 18	7.63E-04
5	30×60	3 Φ 20	9.42E-04	4 Φ 20	1.26E-03
6	30×60	2 Φ 20	6.28E-04	3 Φ 20	9.42E-04
7	30×60	3 Φ 16	6.03E-04	3 Φ 16	6.03E-04
8	30×60	3 Φ 20	9.42E-04	3 Φ 20	9.42E-04
9	30×60	3 Φ 18	7.63E-04	4 Φ 18	1.02E-03
10	30×60	3 Φ 20	9.42E-04	3 Φ 20	9.42E-04
11	30×60	3 Φ 20	9.42E-04	3 Φ 20	9.42E-04
12	30×60	3 Φ 18	7.63E-04	3 Φ 18	7.63E-04
13	30×60	3 Φ 20	9.42E-04	3 Φ 20	9.42E-04

Table 5-10- Detailed Cross sections of the Beams-Part 5

7th Floor(z=+20.40m)

Beam	Section (cm×cm)	Reinforcement			
		Top		Bottom	
		Rebars	Area(m^2)	Rebars	Area(m^2)
1	30×40	3 Φ 14	4.62E-04	3 Φ 14	4.62E-04
2	30×40	2 Φ 20	6.28E-04	2 Φ 14	3.08E-04
3	30×40	3 Φ 14	4.62E-04	3 Φ 14	4.62E-04
4	30×40	3 Φ 14	4.62E-04	3 Φ 14	4.62E-04

Table 5-11- Detailed Cross sections of the Beams-Part 6

5-2) Architectural Drawings:

A) Underground:

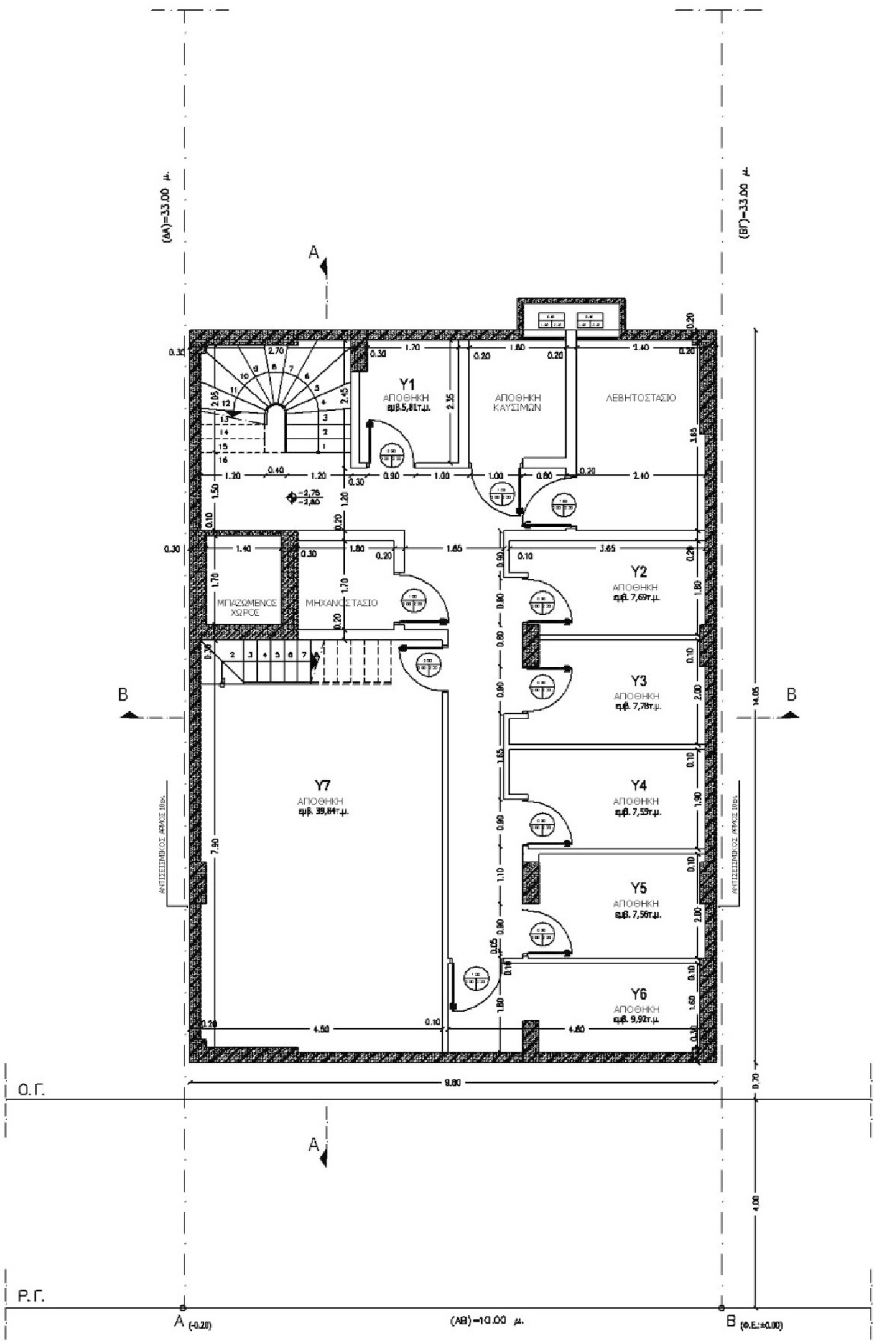

Figure 5-6- Architectural Drawings -Part 1

B) Ground Floor:

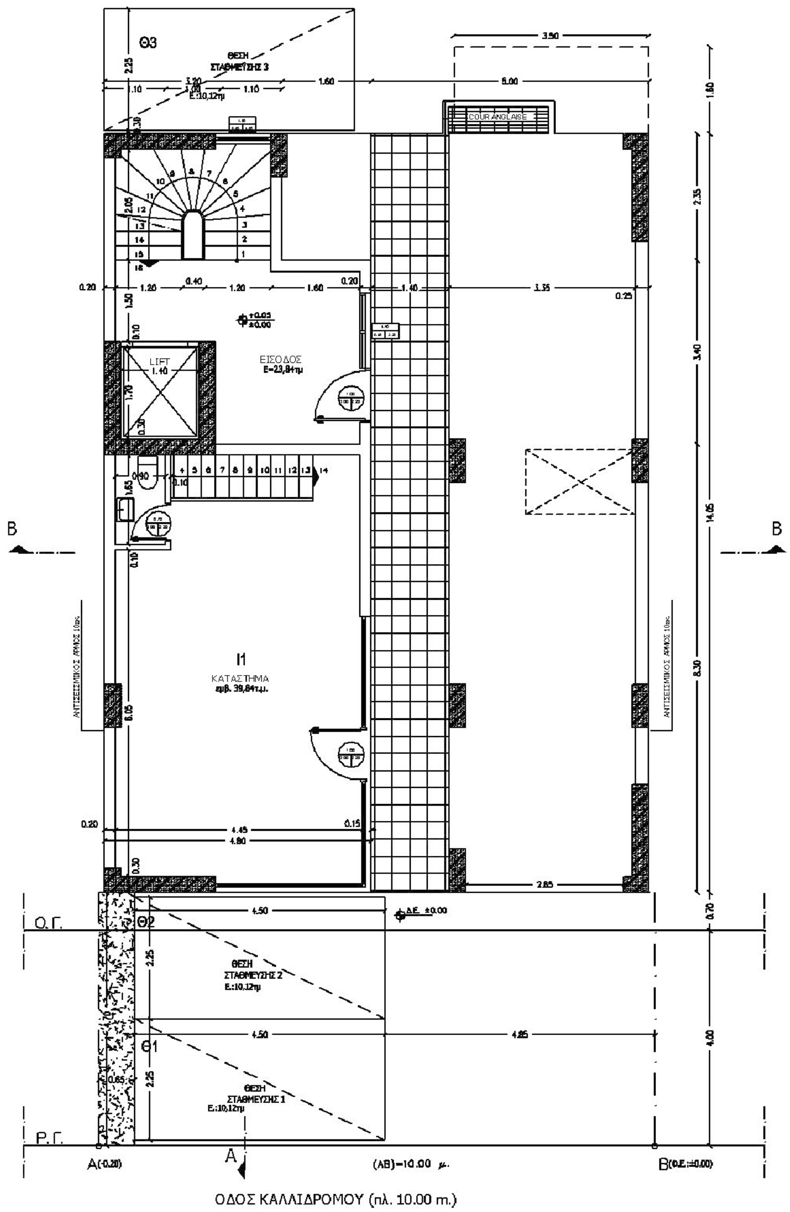

Figure 5-7- Architectural Drawings -Part 2

C) First Floor:

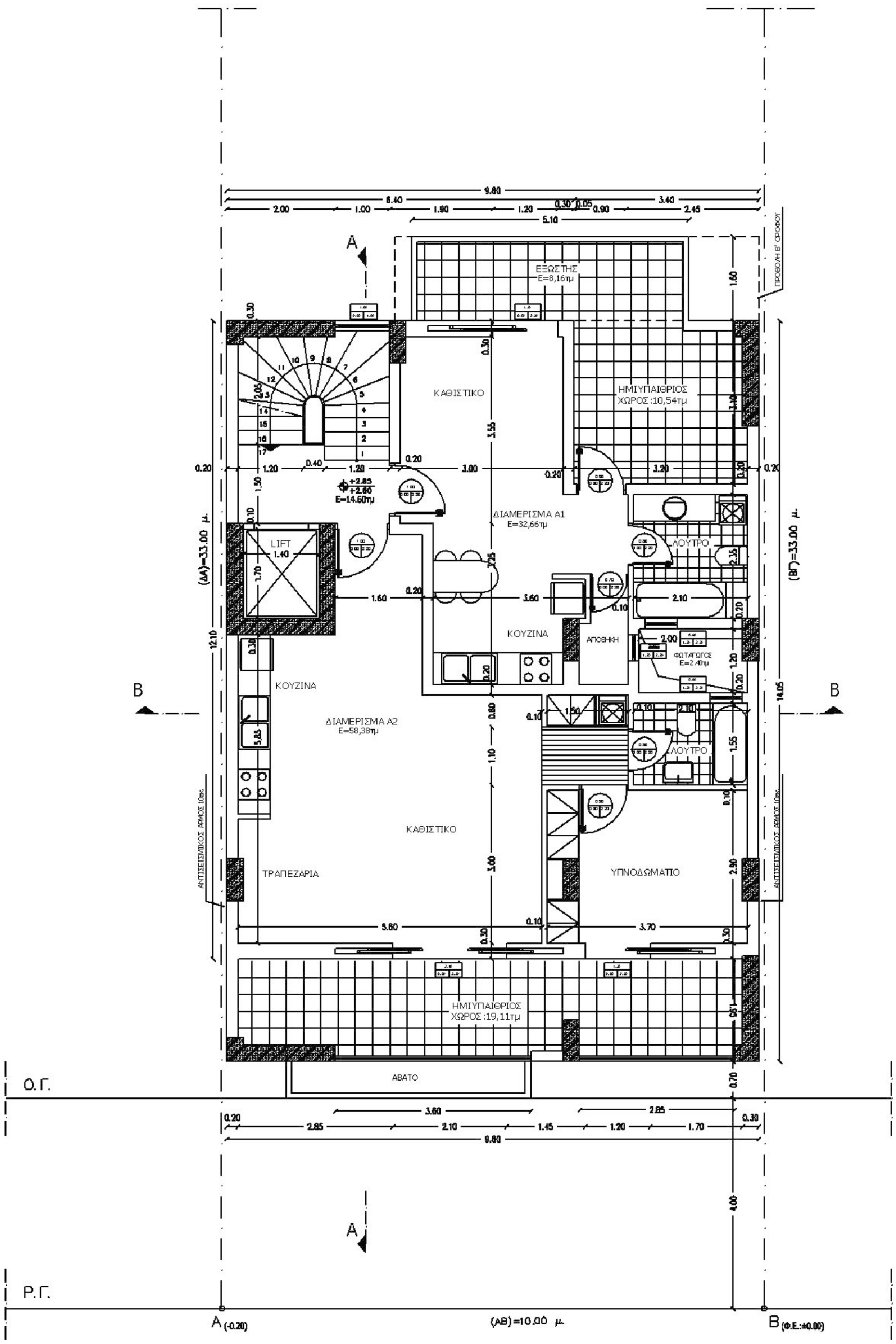

Figure 5-8- Architectural Drawings -Part 3

D) Second Floor:

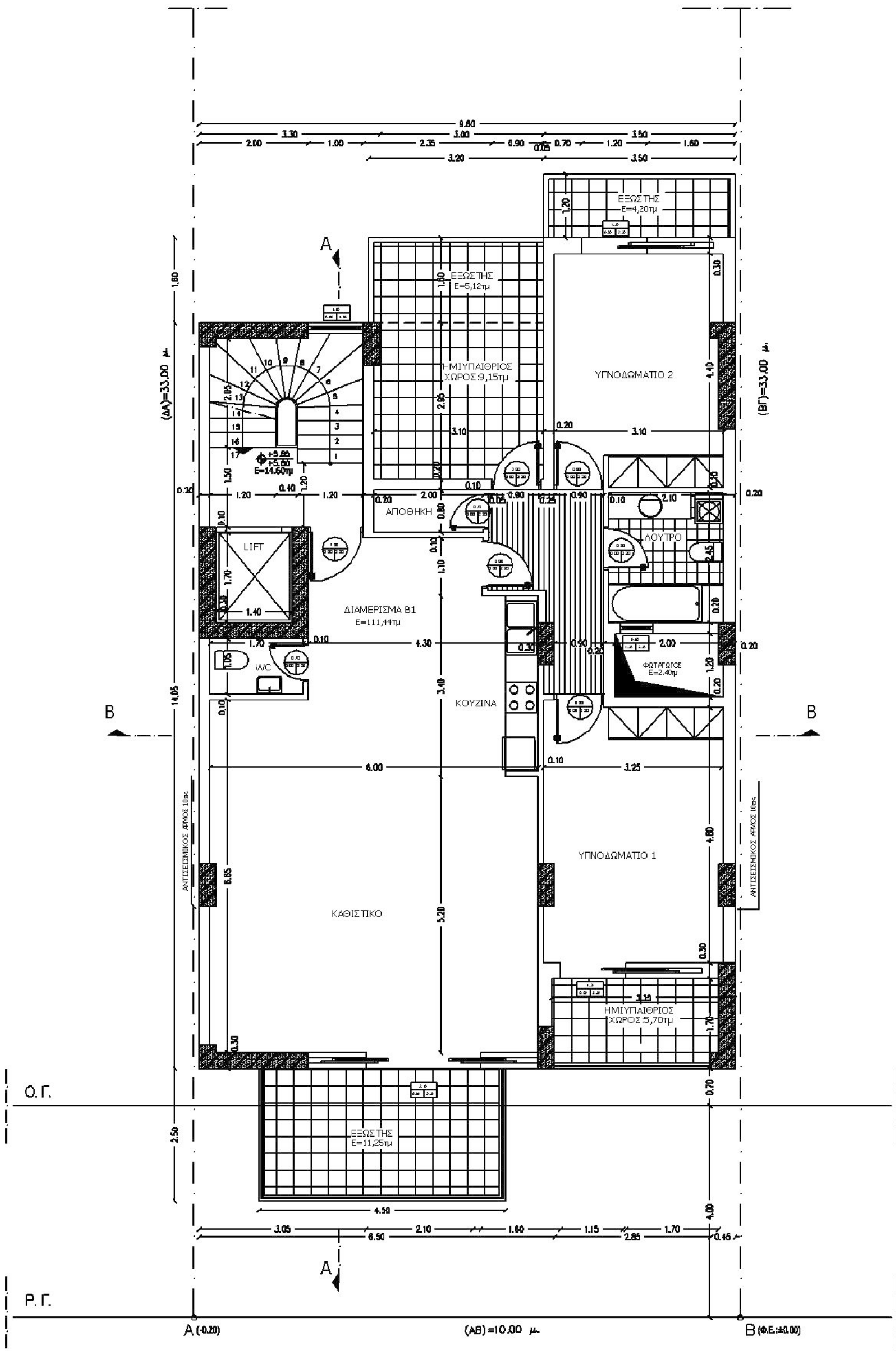

Figure 5-9- Architectural Drawings -Part 4

E) Third Floor:

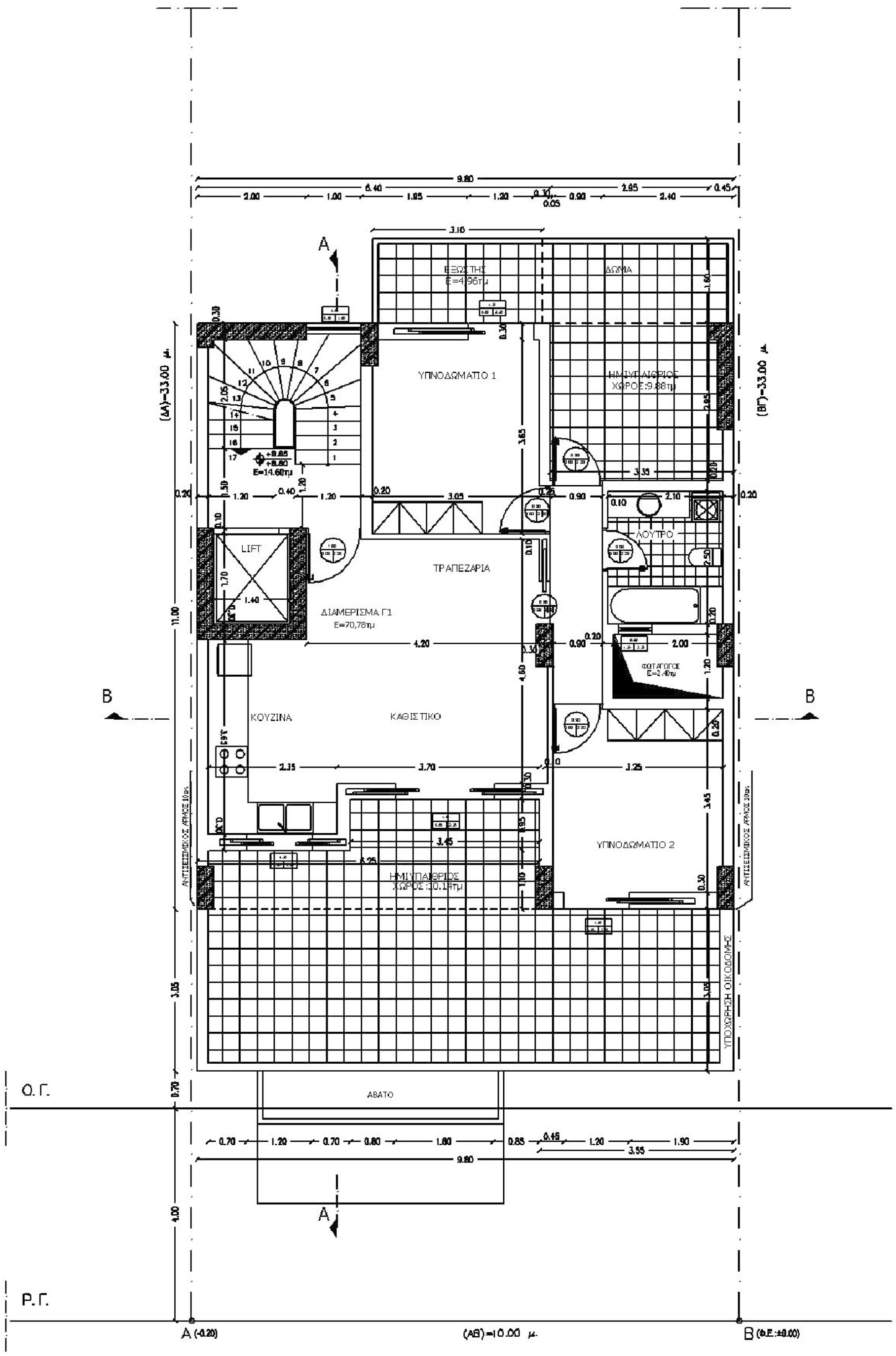

Figure 5-10- Architectural Drawings -Part 5

F) Fourth Floor:

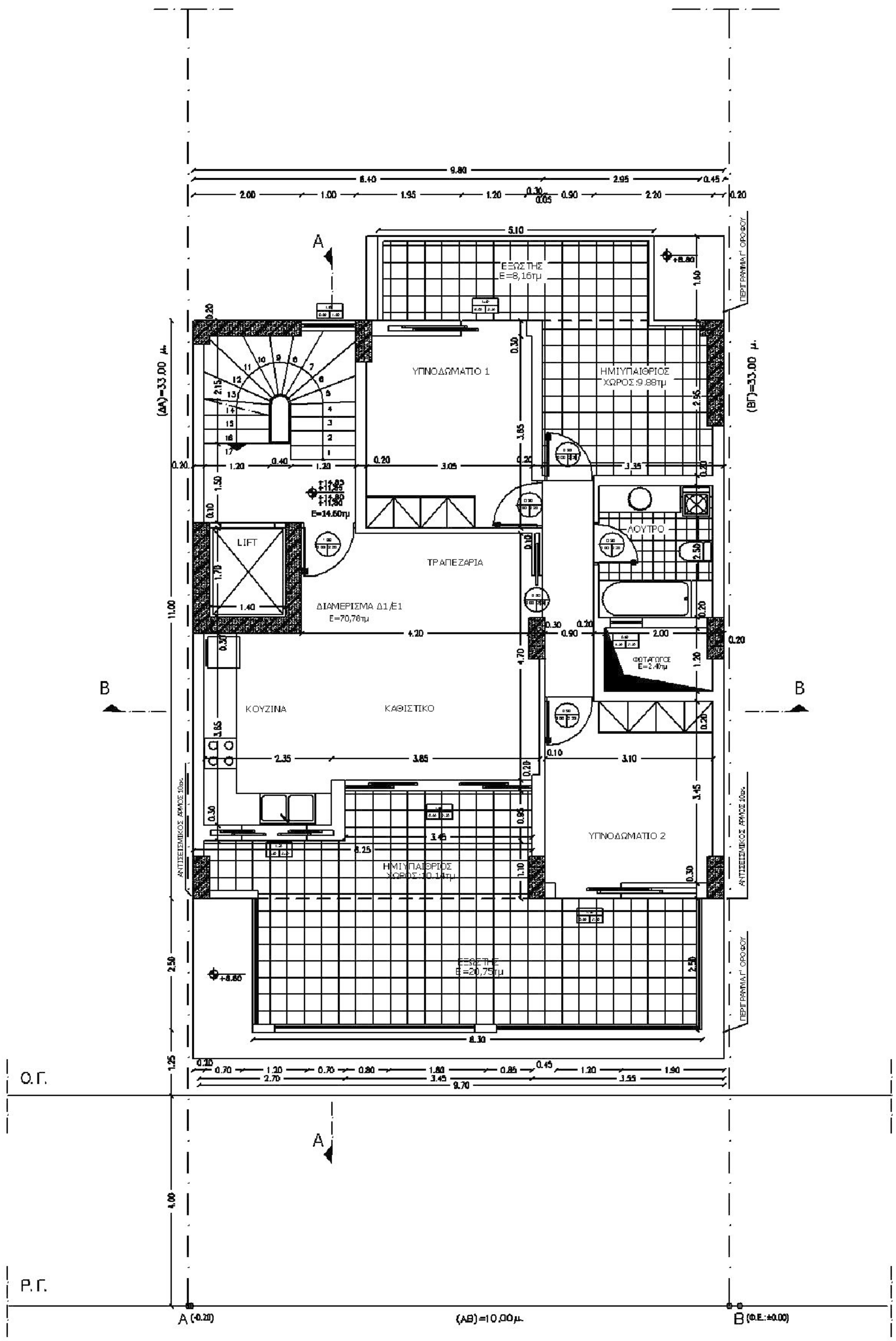

Figure 5-11- Architectural Drawings -Part 6

G) Fifth & Sixth (Lift Room) Floors:

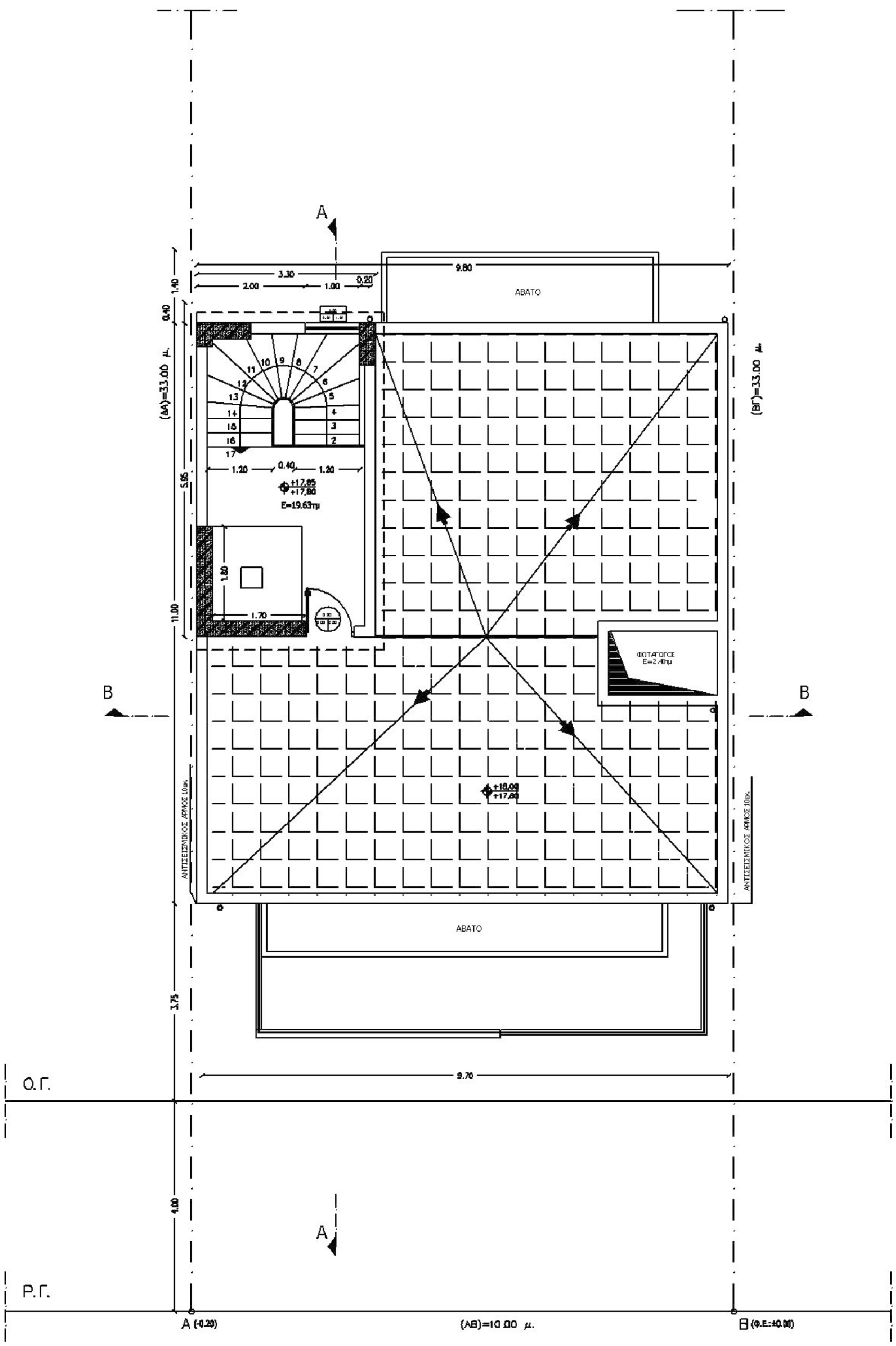

Figure 5-12- Architectural Drawings -Part 7

H) East façade:

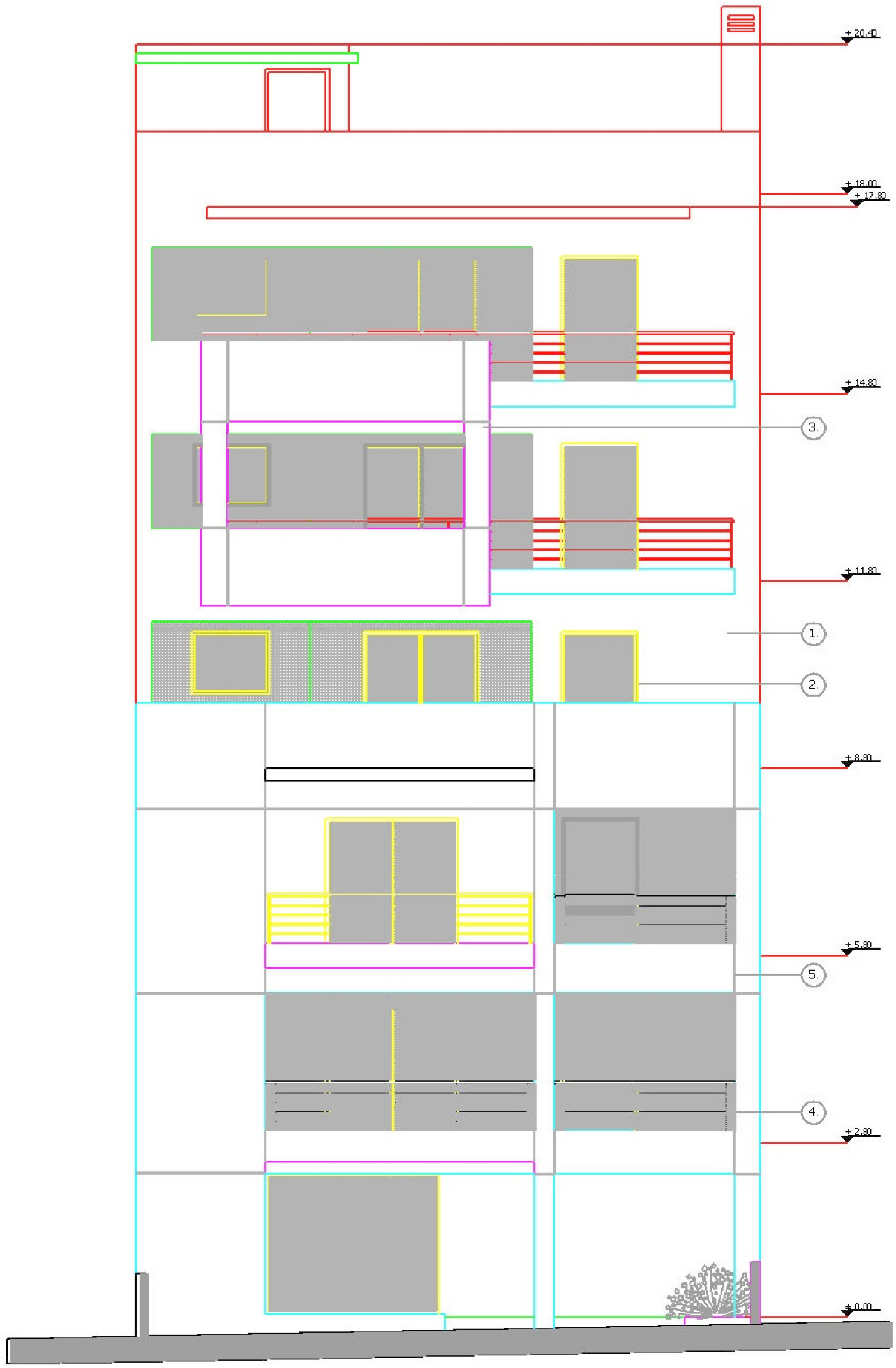

Figure 5-13- Architectural Drawings -Part 8

I) West façade:

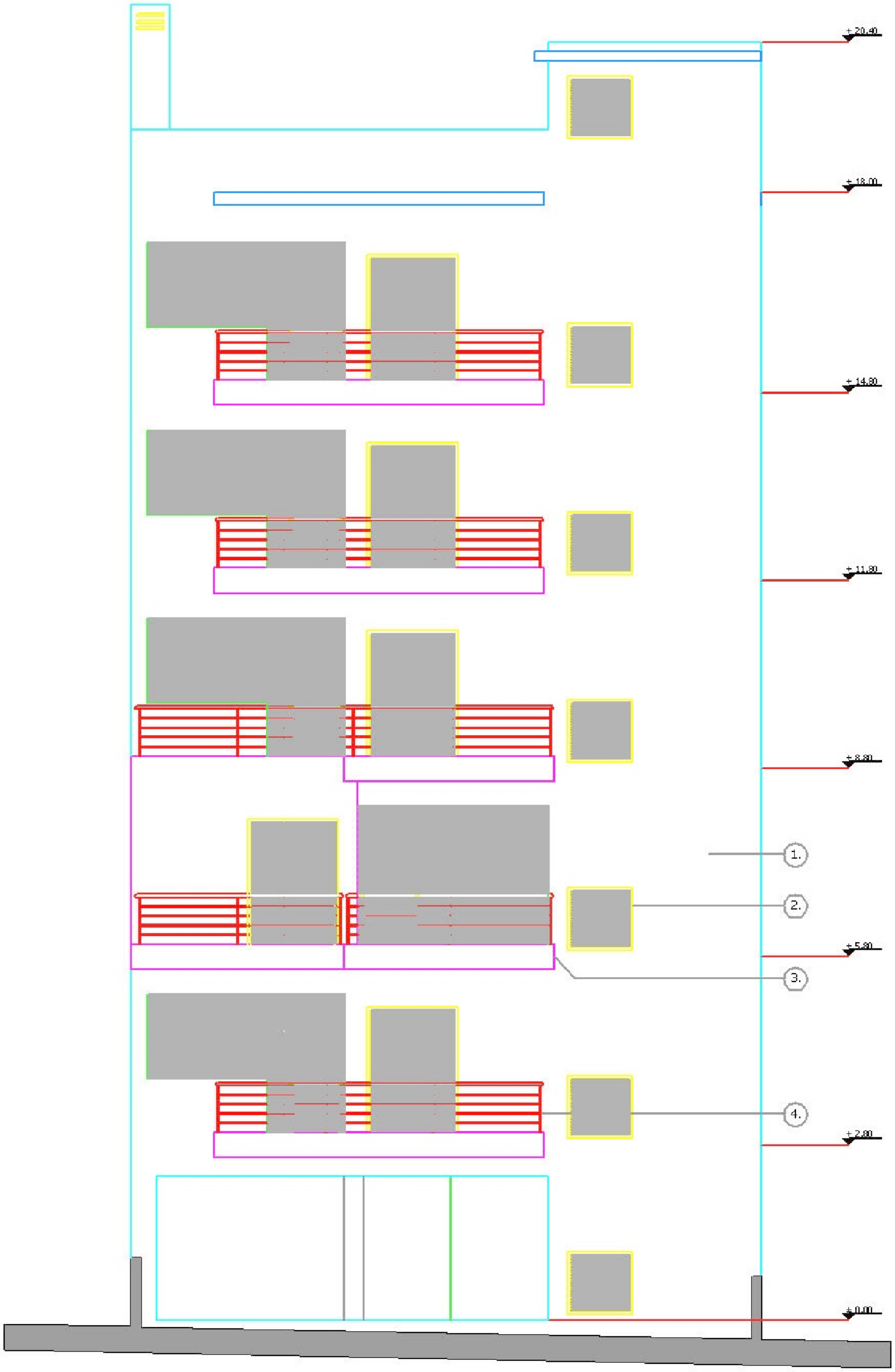

Figure 5-14- Architectural Drawings -Part 9

J) Section A-A:

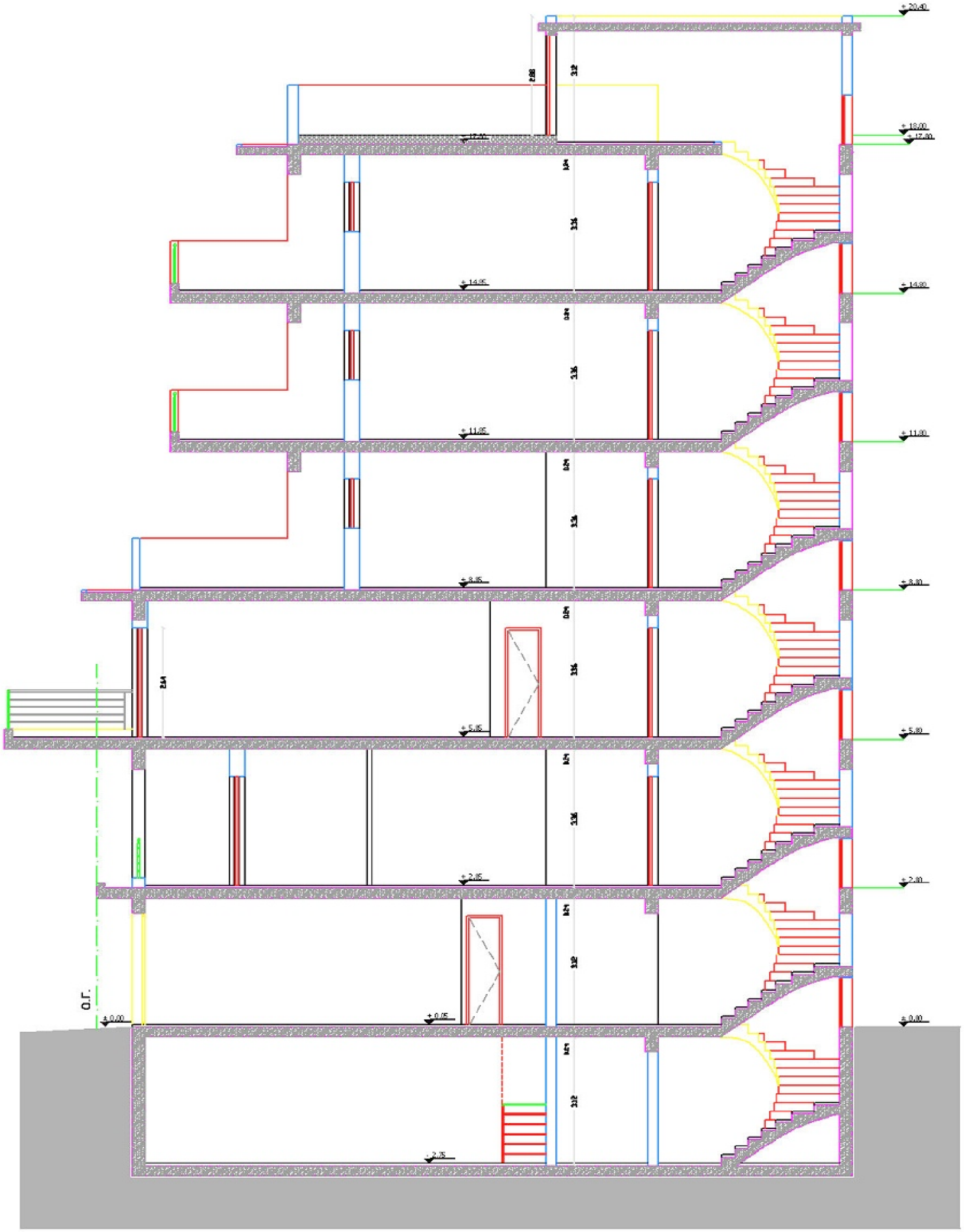

Figure 5-15- Architectural Drawings -Part 10

K) Section B-B:

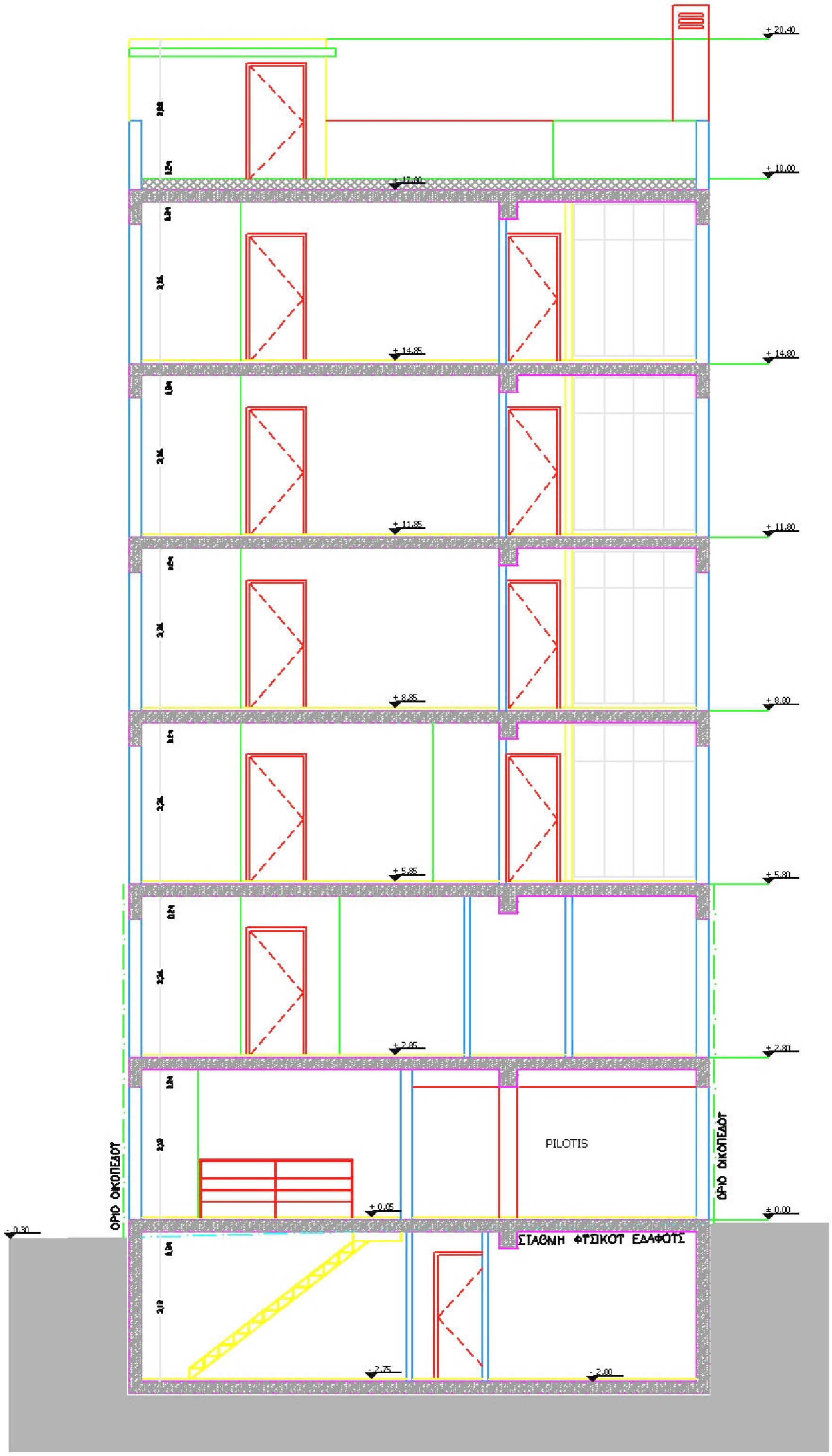

Figure 5-16- Architectural Drawings -Part 11

5-3) Structural Drawings:

A) Underground:

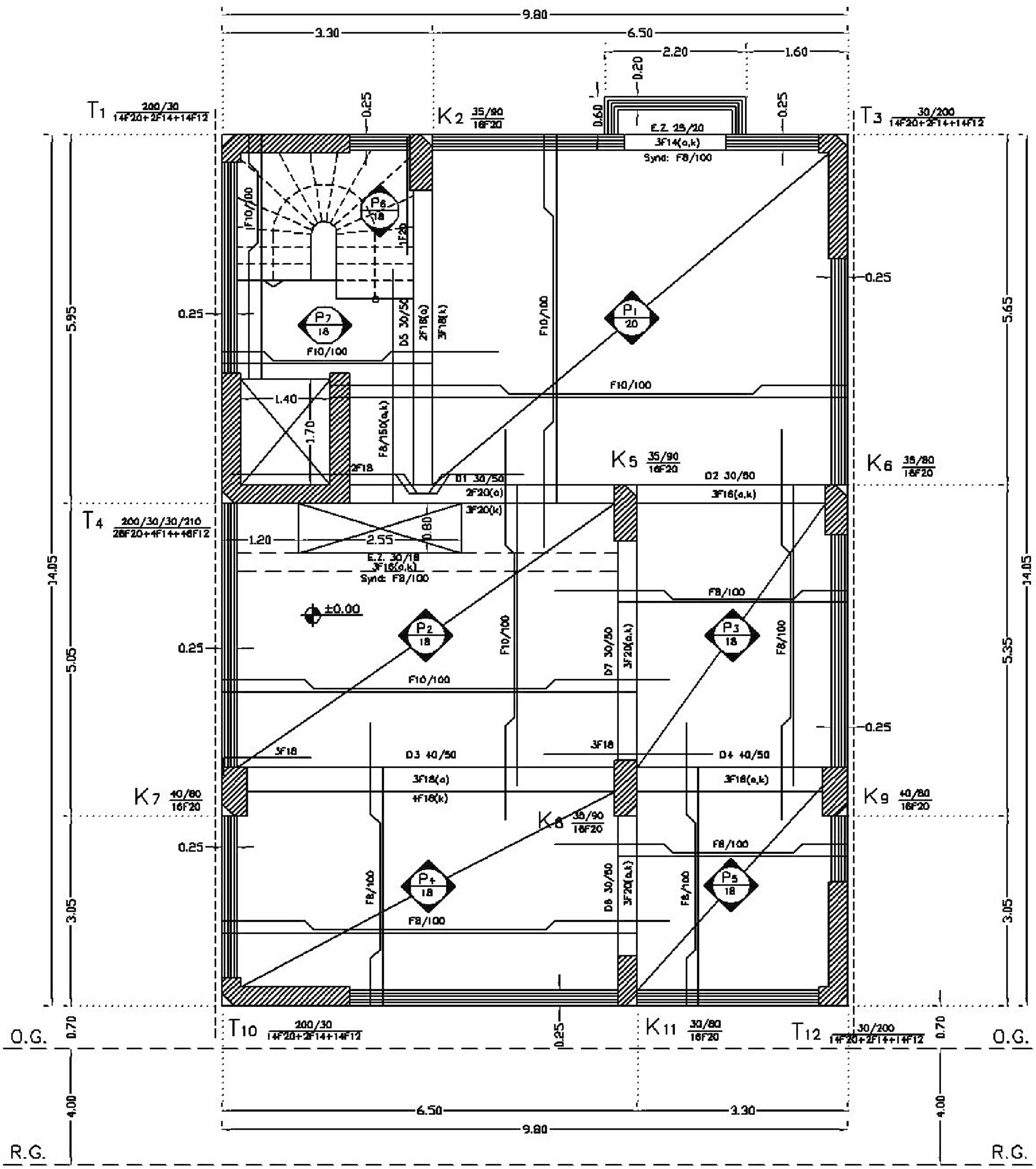

Figure 5-17- Structural Drawings -Part 1

B) Ground Floor:

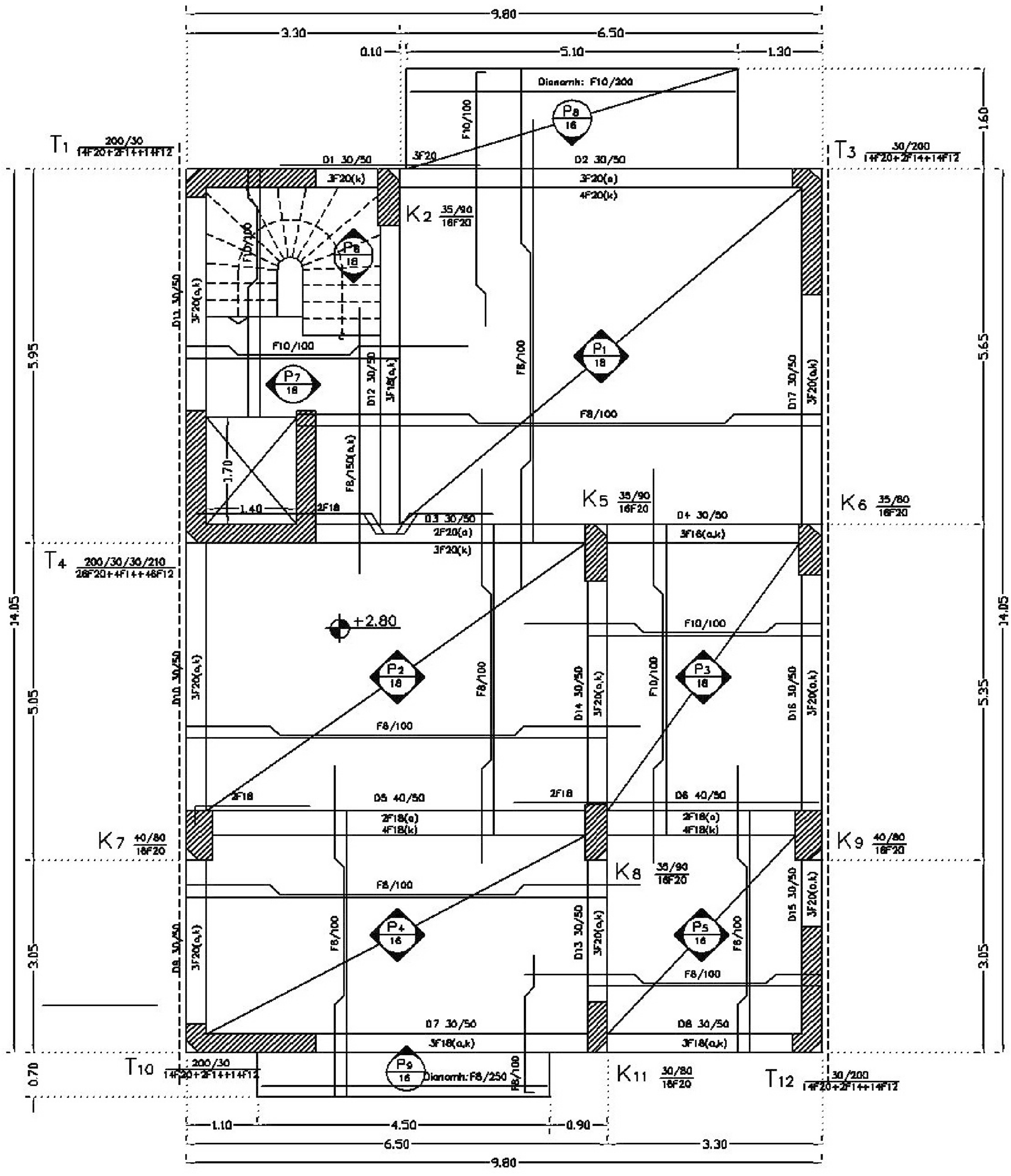

Figure 5-18- Structural Drawings -Part 2

C) First Floor:

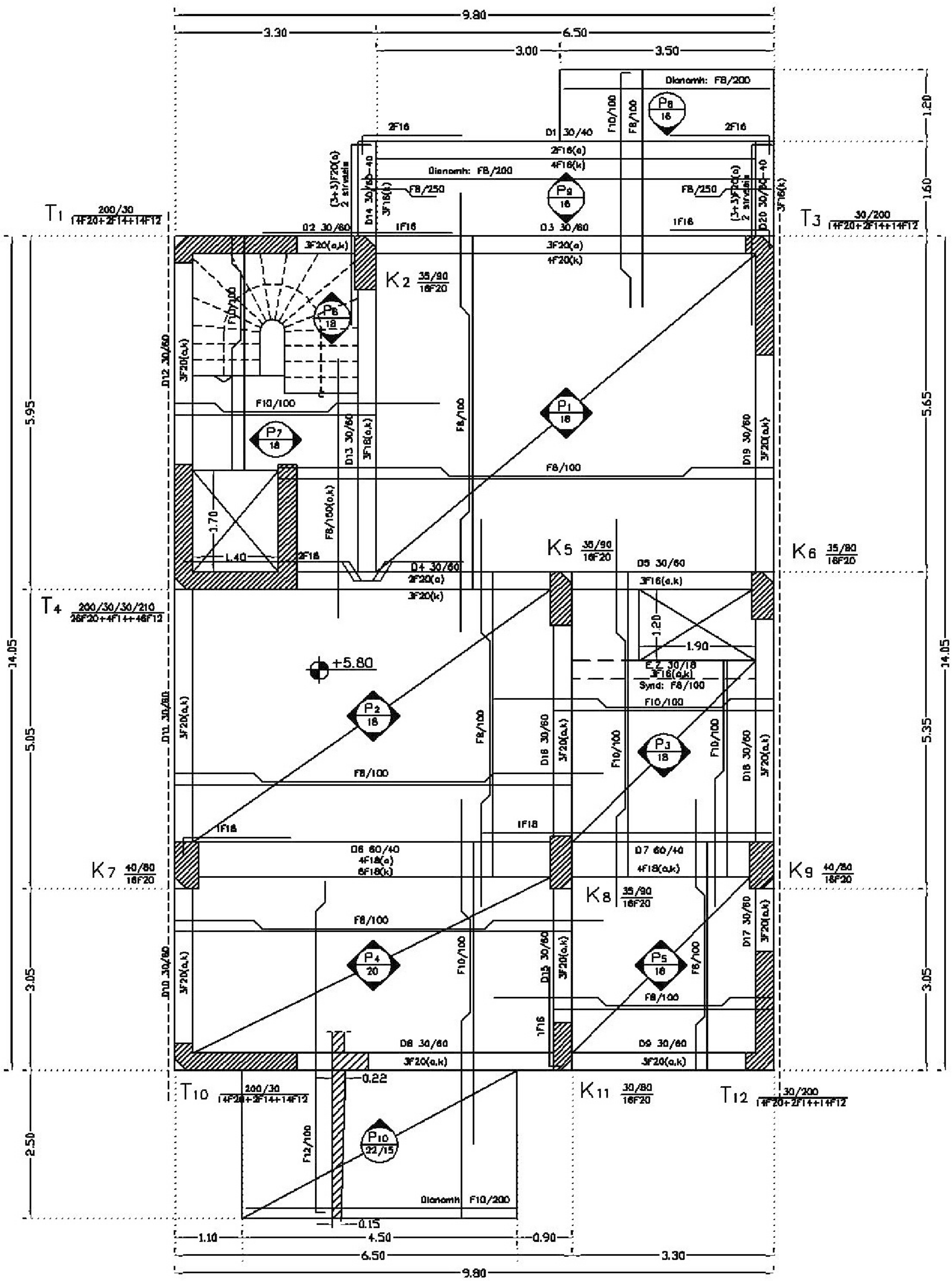

Figure 5-19- Structural Drawings -Part 3

D) Second Floor:

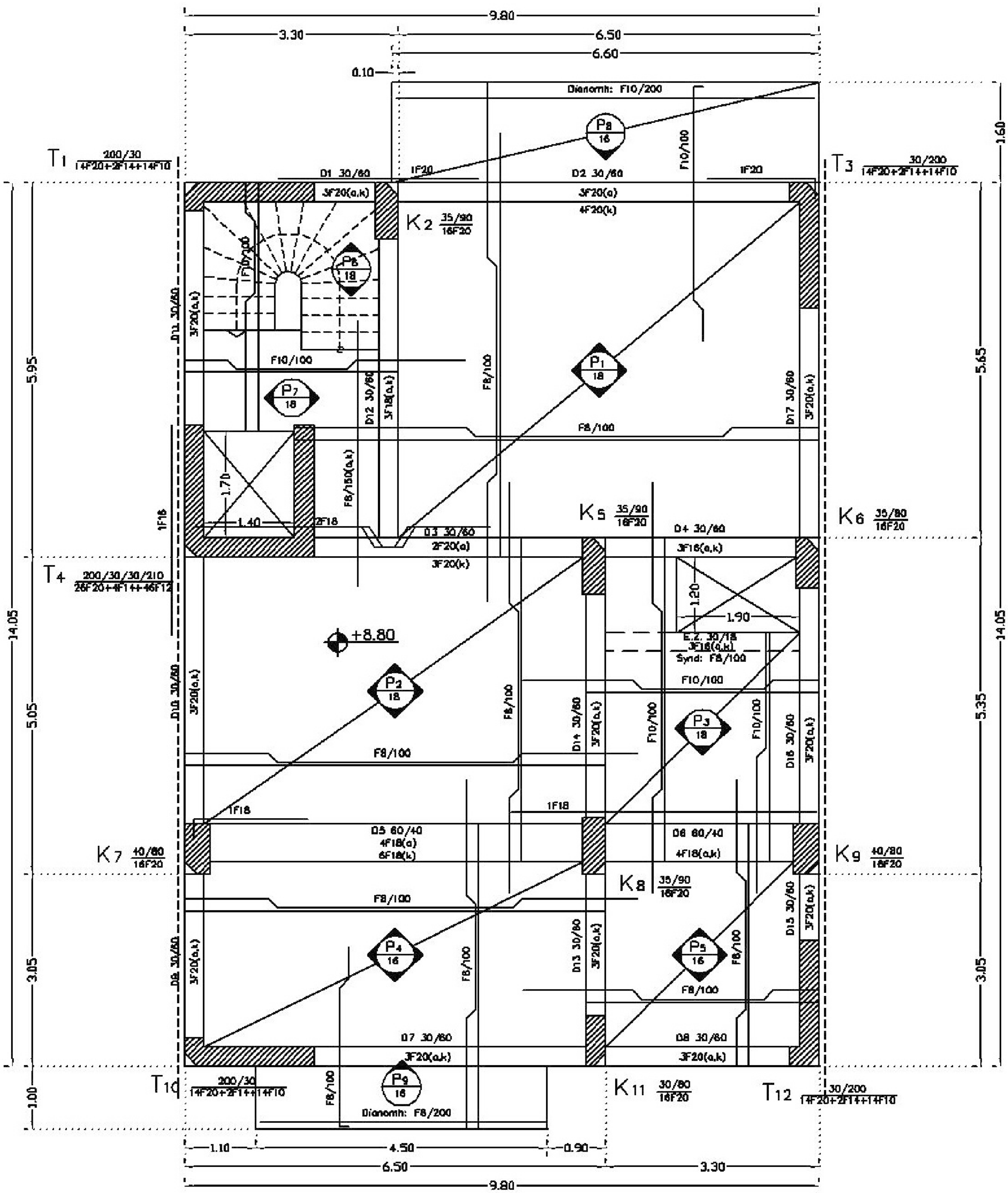

Figure 5-20- Structural Drawings -Part 4

E) Third Floor:

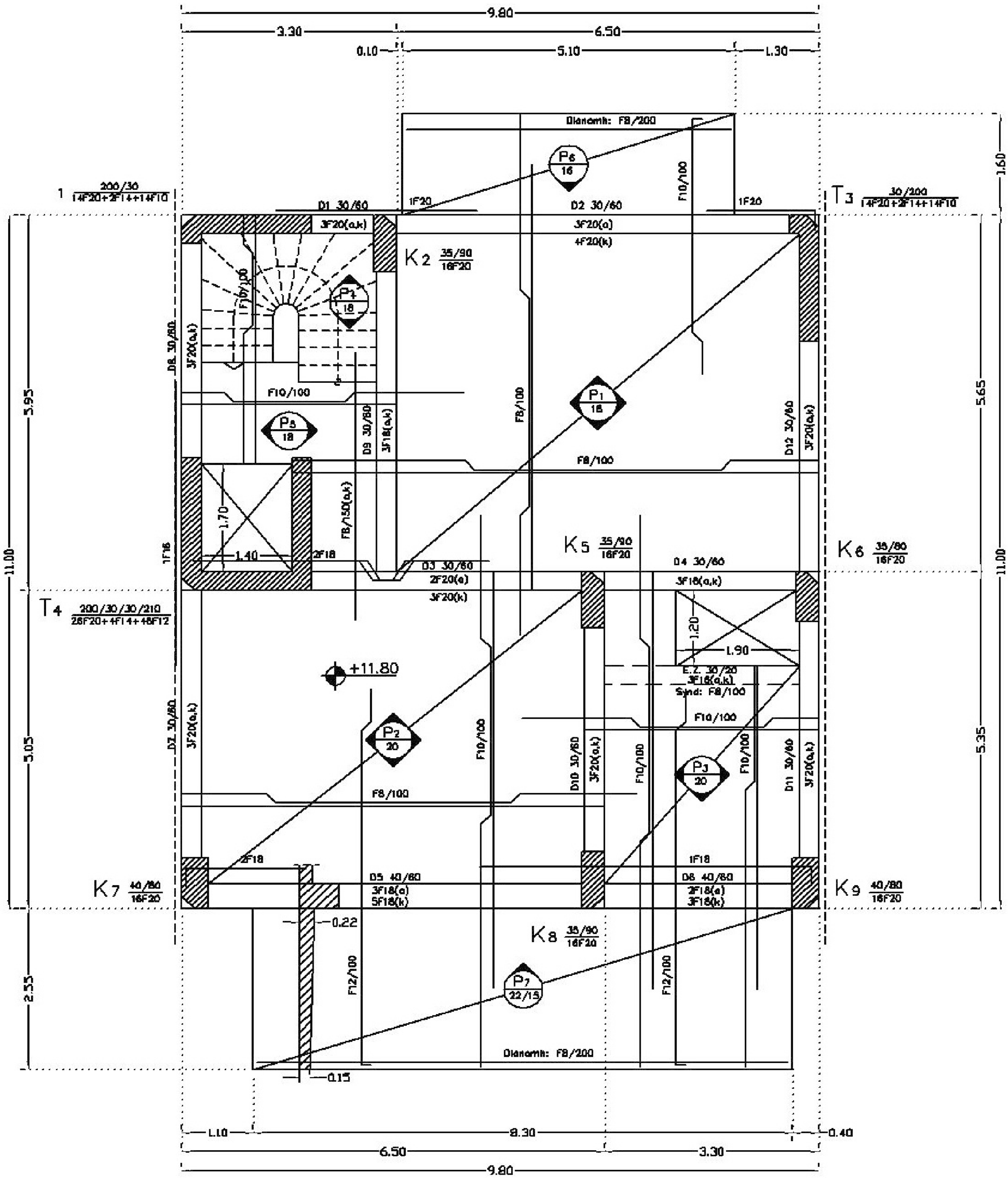

Figure 5-21- Structural Drawings -Part 5

F) Fourth Floor:

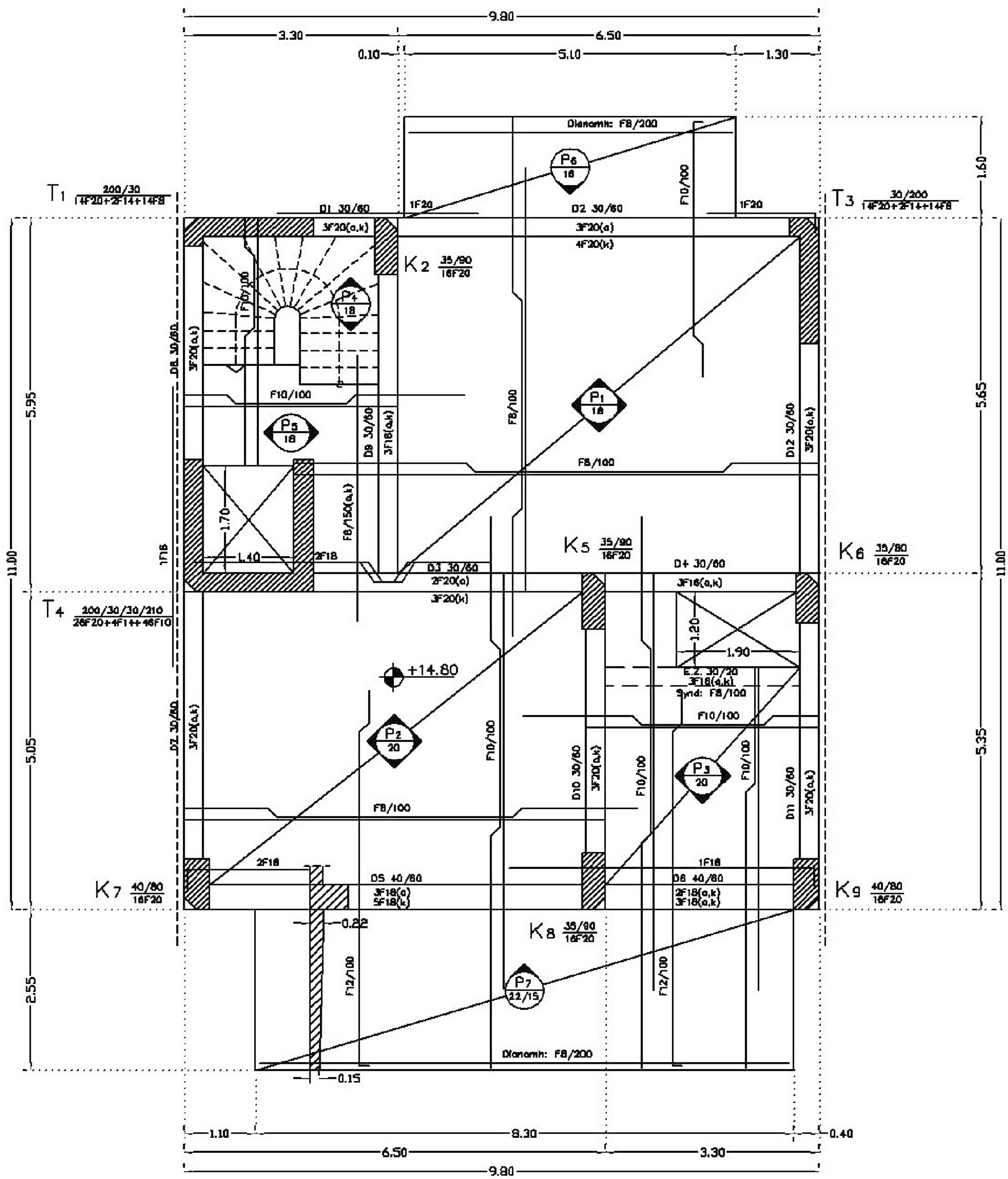

Figure 5-22- Structural Drawings -Part 6

G) Fifth Floor:

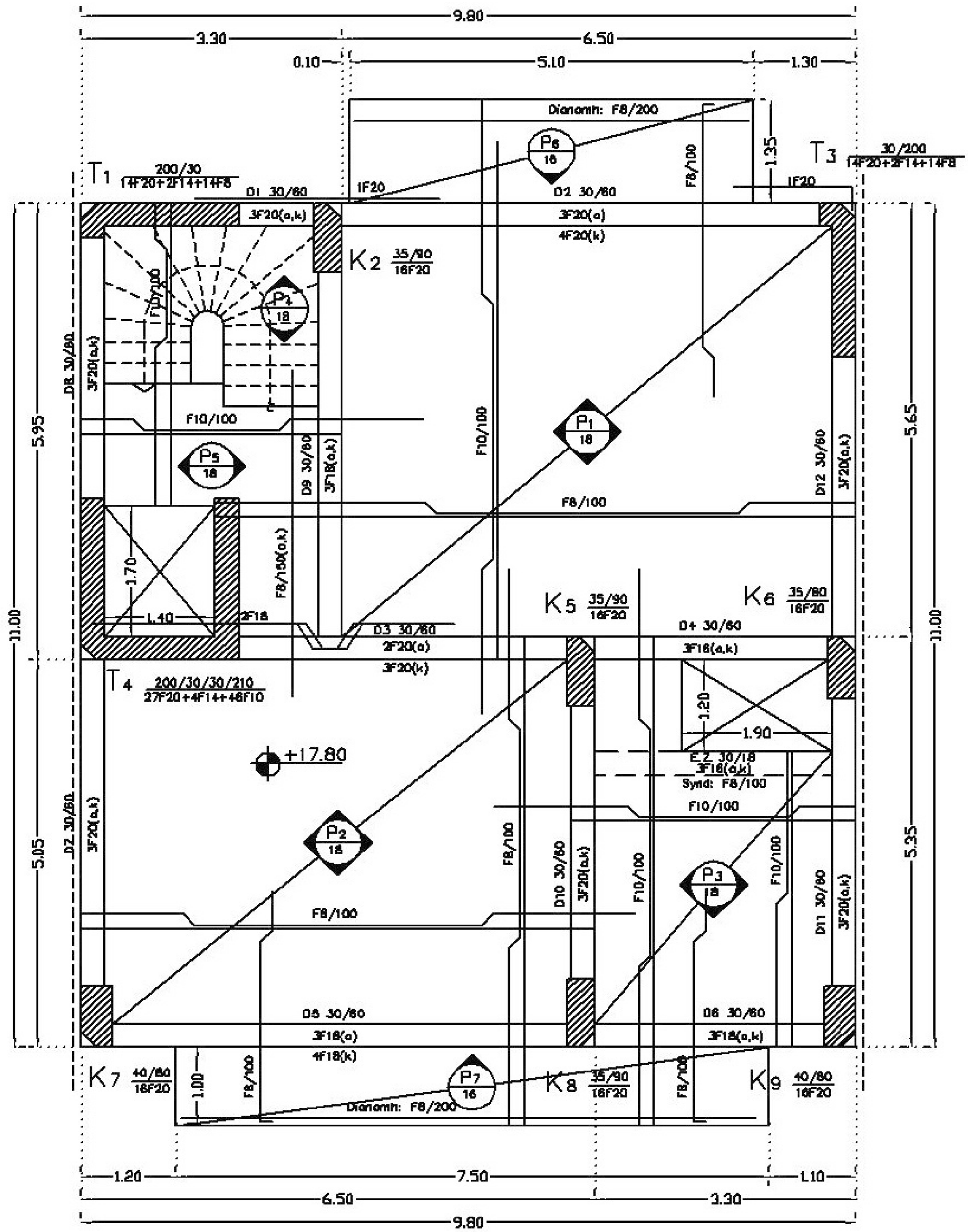

Figure 5-23- Structural Drawings -Part 7

H) Sixth (Lift Room) Floor:

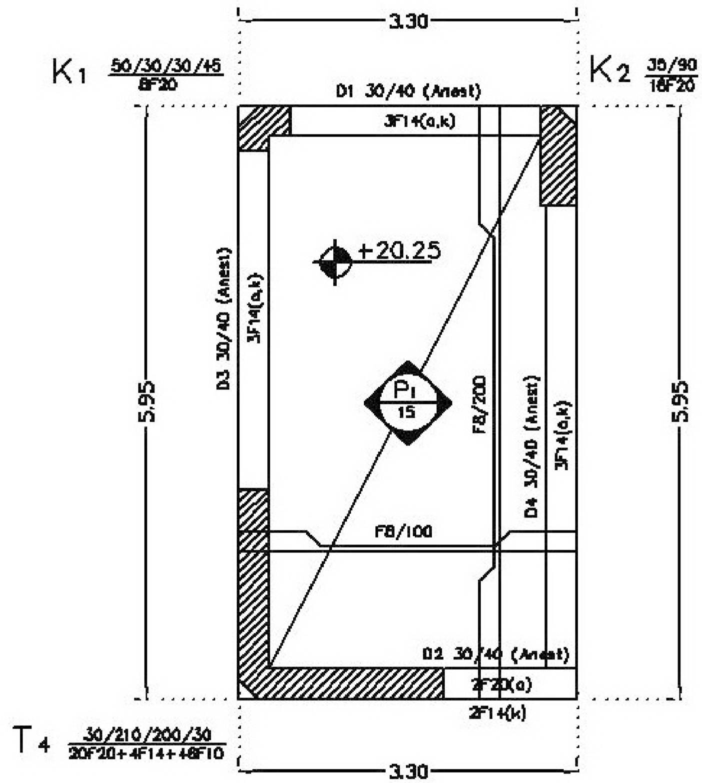

Figure 5-24- Structural Drawings -Part 8

6) Loading:

6-1) Loading Assumptions:

6-1-1) Dead Loads:

Weight of the concrete	25.00	KN/m^3
Weight of the steel	78.50	KN/m^3
Additional dead load	1.50	KN/m^2
Weight of the marbles in the stairs	2.50	KN/m^2
Weight of the Isolation of the roof	2.50	KN/m^2
Weight of the One row brick Partitions(10 cm)	2.10	KN/m^2
Weight of the two rows brick Partitions(20 cm)	3.60	KN/m^2
Weight of the soil	20.00	KN/m^3

Table 6-1- Dead Loads Assumptions

6-1-2) Live Loads:

Main Rooms	2.00	KN/m^2
Shops	5.00	KN/m^2
Offices	3.50	KN/m^2
Parking	5.00	KN/m^2
Balconies	5.00	KN/m^2
Stairs	3.50	KN/m^2
Approachable Roofs	2.00	KN/m^2
Not Approachable Roofs	1.00	KN/m^2
Snow	0.75	KN/m^2
Wind	1.25	KN/m^2

Table 6-2- Live Loads Assumptions

6-1-3) Response spectrum:

Earthquake Combination coefficients	0.30
Ground Acceleration	a = 0.16
Category of the soil	B
Behaviour factor	q = 3.5
Inportance factor	I = 1.00
Damping Ratio	Z = %5

Table 6-3- Response spectrum Assumptions

6-2) Load cases:

6-2-1) Load cases definition:

The Below load cases have been defined for this model:

Case	Type	InitialCond	ModalCase	BaseCase
DEAD	LinStatic	Zero		
MODAL	LinModal	Zero		
Primeter	LinStatic	Zero		
Live	LinStatic	Zero		
Snow	LinStatic	Zero		
Ex	LinRespSpec		MODAL	
Ey	LinRespSpec		MODAL	
Ez	LinRespSpec		MODAL	

Table 6-4- Load Cases definition

6-2-2) Static case load assignments:

Case	LoadType	LoadName	LoadSF
DEAD	Load pattern	DEAD	1.000000
Primeter	Load pattern	Primeter	1.000000
Live	Load pattern	Live	1.000000
Snow	Load pattern	Snow	1.000000

Table 6-5- Static case load assignments

6-2-3) Response spectrum case load assignments:

Case	ModalCombo	GMCf1	GMCf2	PerRigid	DirCombo	DampingType	ConstDamp
		Cyc/sec	Cyc/sec				
Ex	CQC	1.0000E+00	0.0000E+00	SRSS	SRSS	Constant	0.0500
Ey	CQC	1.0000E+00	0.0000E+00	SRSS	SRSS	Constant	0.0500
Ez	CQC	1.0000E+00	0.0000E+00	SRSS	SRSS	Constant	0.0500

Table 6-6- Response spectrum case load assignments-Part 1

Case	LoadType	LoadName	CoordSys	Function	Angle	TransAccSF
					Degrees	m/sec2
Ex	Acceleration	U1	GLOBAL	Eurocode	0.000	1.00000
Ey	Acceleration	U2	GLOBAL	Eurocode	0.000	1.00000
Ez	Acceleration	U3	GLOBAL	Eurocode	0.000	0.15000

Table 6-7- Response spectrum case load assignments- Part 2

6-3) Loading combinations:

Based on the design code, we should consider different load combinations as below:

$$1.35\ \mathrm{D}\ +\ 1.35\ \mathrm{P}\ +\ 1.5\ \mathrm{L} \tag{6.1}$$

$$\mathrm{D}\ +\ \mathrm{P}\ +\ 0.3\ \mathrm{L}\ +\ 0.3\ \mathrm{S} \pm\ \mathrm{E_x} \pm\ 0.3\ \mathrm{E_y} \pm\ 0.3\ \mathrm{E_z} \tag{6.2}$$

$$\mathrm{D}\ +\ \mathrm{P}\ +\ 0.3\ \mathrm{L}\ +\ 0.3\ \mathrm{S} \pm\ 0.3\ \mathrm{E_x} \pm\ \mathrm{E_y} \pm\ 0.3\ \mathrm{E_z} \tag{6.3}$$

$$\mathrm{D}\ +\ \mathrm{P}\ +\ 0.3\ \mathrm{L}\ +\ 0.3\ \mathrm{S} \pm\ 0.3\ \mathrm{E_x} \pm\ 0.3\ \mathrm{E_y} \pm\ \mathrm{E_z} \tag{6.4}$$

6-4) Loads application:

6-4-1) Loading calculations:

6-4-1-1) Ground Floor:

A) Dead Loads:

P1

$\text{Area: } 6.5 \times 5.65 = 36.725\ \mathrm{m^2}$

$\text{Concrete: } 0.25 \times 36.725\ \times 25 = 229.53\ \mathrm{KN}$

$\text{Steel: } (65 \times 5.65 + 57 \times 6.5) \times \pi \times \dfrac{0.01^2}{4}\ \times 78.5 = 4.55\ \mathrm{KN}$

$\text{Additional : } 1.5\ \times 36.725 = 55.09\ \mathrm{KN}$

$\text{Total: } 229.53 + 4.55 + 55.09\ = 289.17\ \mathrm{KN}$

$\dfrac{289.17}{36.725} = 7.87\ \dfrac{\mathrm{KN}}{\mathrm{m^2}}$

P2

$\text{Area: } (5.05 - 0.8) \times (6.5 - 0.25 - 0.3) = 27.97\ \mathrm{m^2}$

$\text{Concrete: } 0.25 \times 27.97 \times 25 = 174.81\ \mathrm{KN}$

Steel: $(47 \times 5.95 + 60 \times 4.7) \times \pi \times \frac{0.01^2}{4} \times 78.5 = 3.46 \text{ KN}$

Additional : $1.5 \times 27.97 = 41.96 \text{ KN}$

Total: $174.81 + 3.46 + 41.96 = 220.23 \text{ KN}$

$$\frac{220.23}{27.97} = 7.87 \ \frac{\text{KN}}{\text{m}^2}$$

P3

Area: $(3.3 - 0.25) \times (5.05 - 0.8) = 12.96 \text{ m}^2$

Concrete: $0.25 \times 12.96 \times 25 = 81 \text{ KN}$

Steel: $(31 \times 4.7 + 47 \times 3.05) \times \pi \times \frac{0.01^2}{4} \times 78.5 = 1.78 \text{ KN}$

Additional : $1.5 \times 12.96 = 19.44 \text{ KN}$

Total: $81 + 1.78 + 19.44 = 102.22 \text{ KN}$

$$\frac{102.22}{12.96} = 7.87 \ \frac{\text{KN}}{\text{m}^2}$$

P4

Area: $(6.5 - 0.25 - 0.3) \times (3.05 + 0.8 - 0.25 - 0.4) = 19.04 \text{ m}^2$

Concrete: $0.25 \times 19.04 \times 25 = 119 \text{ KN}$

Steel: $(60 \times 3.2 + 32 \times 5.95) \times \pi \times \frac{0.01^2}{4} \times 78.5 = 2.36 \text{ KN}$

Additional : $1.5 \times 19.04 = 28.56 \text{ KN}$

Total: $119 + 2.36 + 28.56 = 149.92 \text{ KN}$

$$\frac{149.92}{19.04} = 7.87 \ \frac{\text{KN}}{\text{m}^2}$$

P5

Area: $(3.3 - 0.25) \times (3.05 + 0.8 - 0.25 - 0.4) = 9.76 \text{ m}^2$

Concrete: $0.25 \times 9.76 \times 25 = 61 \text{ KN}$

Steel: $(31 \times 3.2 + 32 \times 3.05) \times \pi \times \frac{0.01^2}{4} \times 78.5 = 1.21 \text{ KN}$

Additional : $1.5 \times 9.76 = 14.64 \text{ KN}$

Total: $61 + 1.21 + 14.64 = 76.85 \text{ KN}$

$$\frac{76.85}{9.76} = 7.87 \frac{\text{KN}}{\text{m}^2}$$

P6,P7

Area: $3.3 \times 5.95 - 1.4 \times 1.7 = 17.255 \text{ m}^2$

Concrete: $0.25 \times 17.255 \times 25 = 107.84 \text{ KN}$

Steel: $(33 \times 5.05 + 60 \times 3.3) \times \pi \times \frac{0.01^2}{4} \times 78.5 = 2.25 \text{ KN}$

Marbles: $2.5 \times 17.255 = 43.14 \text{ KN}$

Total: $107.84 + 2.25 + 43.14 = 153.23 \text{ KN}$

$$\frac{153.23}{17.255} = 8.88 \frac{\text{KN}}{\text{m}^2}$$

Equivalent dead load of the partitions

$14.05 \times 2.4 \times 3.6 = 121.392 \text{ KN}$

$9.8 \times 14.05 - (5.95 \times 3.3) = 118.055 \text{ m}^2$

$$\frac{121.392}{118.055} = 1.03 \frac{\text{KN}}{\text{m}^2}$$

Walls of the perimeter

$$3.6 \times (2.8 - 0.5) = 8.28\ \frac{\text{KN}}{\text{m}}$$

B) Live Loads:

$$\text{Total of the plan: } 5\ \frac{\text{KN}}{\text{m}^2}$$

$$\text{Stairs: } 3.5\ \frac{\text{KN}}{\text{m}^2}$$

6-4-1-2) First Floor:

A) Dead Loads:

P1

$$\text{Area of Isolation: } 3.2 \times 3.1 + 2.1 \times 2.35 = 14.86\ \text{m}^2$$

$$\text{Steel: } (65 \times 5.65 + 57 \times 6.5) \times \pi \times \frac{0.008^2}{4} \times 78.5 = 2.91\ \text{KN}$$

$$\text{Total: } 229.53 + 2.91 + 55.09 = 287.53\ \text{KN}$$

$$\frac{287.53}{36.725} + \frac{1 \times 14.86}{36.725} = 8.23\ \frac{\text{KN}}{\text{m}^2}$$

P2

$$\text{Steel: } (47 \times 5.95 + 60 \times 4.7) \times \pi \times \frac{0.008^2}{4} \times 78.5 = 2.22\ \text{KN}$$

$$\text{Total: } 174.81 + 2.22 + 41.96 = 218.99\ \text{KN}$$

$$\frac{218.99}{27.97} = 7.83\ \frac{\text{KN}}{\text{m}^2}$$

P3

Area of Isolation: $1.55 \times 2.1 = 3.26\ \text{m}^2$

$$7.87 + \frac{1 \times 3.26}{12.96} = 8.12\ \frac{\text{KN}}{\text{m}^2}$$

P4

Area of Isolation: $6 \times 1.95 = 11.7\ \text{m}^2$

$$\text{Steel: } (60 \times 3.2 + 32 \times 5.95) \times \pi \times \frac{0.008^2}{4} \times 78.5 = 1.51\ \text{KN}$$

$$\text{Total: } 119 + 1.51 + 28.56 = 149.07\ \text{KN}$$

$$\frac{149.07}{19.04} + \frac{1 \times 11.7}{19.04} = 8.44\ \frac{\text{KN}}{\text{m}^2}$$

P5

Area of Isolation: $3.05 \times 1.95 = 5.95\ \text{m}^2$

$$\text{Steel: } (31 \times 3.2 + 32 \times 3.05) \times \pi \times \frac{0.008^2}{4} \times 78.5 = 0.78\ \text{KN}$$

$$\text{Total: } 61 + 0.78 + 14.64 = 76.42\ \text{KN}$$

$$\frac{76.85}{9.76} + \frac{1 \times 5.95}{9.76} = 8.44\ \frac{\text{KN}}{\text{m}^2}$$

P6,P7

$$8.88\ \frac{\text{KN}}{\text{m}^2}$$

Equivalent dead load of the partitions

$$22 \times 2.6 \times 3.6 + 9 \times 2.6 \times 2.1 = 255.06\ \text{KN}$$

$$9.8\times 14.05-(5.95\times 3.3)=118.055\ \mathrm{m^2}$$

$$\frac{255.06}{118.055}=2.16\ \frac{\mathrm{KN}}{\mathrm{m^2}}$$

Walls of the perimeter

$$3.6\times(3-0.5)=9\ \frac{\mathrm{KN}}{\mathrm{m}}$$

B) Live Loads:

$$\text{Total of the plan: } 2\ \frac{\mathrm{KN}}{\mathrm{m^2}}$$

$$\text{Stairs: } 3.5\ \frac{\mathrm{KN}}{\mathrm{m^2}}$$

C) Balconies:

Lower balcony:

Dead load:

$$\text{Diaphragm: } 4.3\times 0.7\ \times 7.83=23.57\ \mathrm{KN}$$

$$\text{Walls: } 2\times 0.7\times 2.6\times 2.1=7.64\ \mathrm{KN}$$

$$\text{Total: } 23.57+7.64=31.21\ \mathrm{KN}$$

$$\frac{31.21}{(6.5-2\times 0.3)}=5.29\ \frac{\mathrm{KN}}{\mathrm{m}}$$

Dead Moment:

$$\frac{31.21}{4.3}=7.26\ \frac{\mathrm{KN}}{\mathrm{m}}$$

$$7.26\ \times\frac{0.7^2}{2}=1.78\ \mathrm{KN.m}$$

Live load:

$$4.3 \times 0.7 \times 5 = 15.05 \text{ KN}$$

$$\frac{15.05}{(6.5 - 2 \times 0.3)} = 2.55 \ \frac{\text{KN}}{\text{m}}$$

Live Moment:

$$\frac{15.05}{4.3} = 3.5 \ \frac{\text{KN}}{\text{m}}$$

$$3.5 \ \times \frac{0.7^2}{2} = 0.86 \text{ KN.m}$$

Upper balcony:

Dead load:

Area of Isolation: $5.1 \times 1.6 = 8.16 \text{ m}^2$

Diaphragm: $5.1 \times 1.6 \times 7.83 = 63.89 \text{ KN}$

Walls: $2 \times 1.6 \times 2.6 \times 2.1 = 17.47 \text{ KN}$

Isolation: $8.16 \times 1 = 8.16 \text{ KN}$

Total: $63.89 + 17.47 + 8.16 = 89.52 \text{ KN}$

$$\frac{89.52}{6.15} = 14.56 \ \frac{\text{KN}}{\text{m}}$$

Dead Moment:

$$\frac{89.52}{5.1} = 17.55 \ \frac{\text{KN}}{\text{m}}$$

$$17.55 \ \times \frac{1.6^2}{2} = 22.46 \text{ KN.m}$$

Live load:

$$5.1 \times 1.6 \times 5 = 40.8 \text{ KN}$$

$$\frac{40.8}{6.15} = 6.63 \ \frac{\text{KN}}{\text{m}}$$

Live Moment:

$$\frac{40.8}{5.1} = 8 \ \frac{\text{KN}}{\text{m}}$$

$$8 \times \frac{1.6^2}{2} = 10.24 \text{ KN.m}$$

6-4-1-3) Second Floor:

A) Dead Loads:

P1

Area of Isolation: $2.95 \times 3.1 + 2.45 \times 2.1 = 14.29 \text{ m}^2$

$$7.83 + \frac{1 \times 14.29}{36.725} = 8.22 \ \frac{\text{KN}}{\text{m}^2}$$

P2

$$\frac{218.99}{27.97} = 7.83 \ \frac{\text{KN}}{\text{m}^2}$$

P3

Area: $12.96 - (1.2 \times 1.9) = 10.68 \text{ m}^2$

$$\frac{102.22}{10.68} = 9.57 \ \frac{\text{KN}}{\text{m}^2}$$

P4

$$\text{Steel:}\left((60\times 3.2)\times\pi\times\frac{0.01^2}{4}+(32\times 5.95)\times\pi\times\frac{0.008^2}{4}\right)\times 78.5 = 1.94\ \text{KN}$$

$$\text{Total: } 119 + 1.94 + 28.56 = 149.5\ \text{KN}$$

$$\frac{149.5}{19.04} = 7.85\ \frac{\text{KN}}{\text{m}^2}$$

P5

$$\text{Area of Isolation: } 3.35\ \times 1.7 = 5.7\ \text{m}^2$$

$$7.83 + \frac{1\times 5.7}{9.76} = 8.41\ \frac{\text{KN}}{\text{m}^2}$$

P6,P7

$$8.88\ \frac{\text{KN}}{\text{m}^2}$$

Equivalent dead load of the partitions

$$13.5\times 2.6\times 3.6 + 15\times 2.6\times 2.1 = 208.26\ \text{KN}$$

$$\text{Area: } 118.055 - 1.2\times 1.9 = 115.78\ \text{m}^2$$

$$\frac{208.26}{115.78} = 1.8\ \frac{\text{KN}}{\text{m}^2}$$

Walls of the perimeter

$$3.6\times(3-0.5) = 9\ \frac{\text{KN}}{\text{m}}$$

B) Live Loads:

$$\text{Total of the plan: } 2\ \frac{\text{KN}}{\text{m}^2}$$

$$\text{Stairs: } 3.5\ \frac{\text{KN}}{\text{m}^2}$$

C) Balconies:

Lower balcony:

Dead load:

$$\text{Diaphragm: } 4.5 \times 2.5\ \times 7.83 = 88.09\ \text{KN}$$

$$\text{Additional Isolation: } 4.5 \times 2.5 \times 1 = 11.25\ \text{KN}$$

$$\text{Total: } 88.09\ + 11.25 = 99.34\ \text{KN}$$

$$\frac{99.34}{5.9} = 16.84\ \frac{\text{KN}}{\text{m}}$$

Dead Moment:

$$\frac{99.34}{4.5} = 22.07\ \frac{\text{KN}}{\text{m}}$$

$$22.07\ \times \frac{2.5^2}{2} = 68.97\ \text{KN.m}$$

Live load:

$$4.5 \times 2.5 \times 5 = 56.25\ \text{KN}$$

$$\frac{56.25}{5.9} = 9.53\ \frac{\text{KN}}{\text{m}}$$

Live Moment:

$$\frac{56.25}{4.5} = 12.5\ \frac{\mathrm{KN}}{\mathrm{m}}$$

$$12.5\ \times \frac{2.5^2}{2} = 39.06\ \mathrm{KN.m}$$

Upper balcony:

Dead load:

Area of Isolation: $3.2 \times 1.6 + 3.5 \times 1.2 = 9.32\ \mathrm{m}^2$

Diaphragm: $6.85 \times 1.6 \times 7.83 = 85.82\ \mathrm{KN}$

$3.5 \times 1.2 \times 7.83 = 32.89\ \mathrm{KN}$

Isolation: $9.32 \times 1 = 9.32\ \mathrm{KN}$

Walls: $2 \times 1.6 \times 2.6 \times 2.1 = 17.47\ \mathrm{KN}$

Total: $85.82 + 32.89 + 9.32 + 17.47 = 145.5\ \mathrm{KN}$

$$\frac{145.5}{6.15} = 23.66\ \frac{\mathrm{KN}}{\mathrm{m}}$$

Dead Moment:

$$\frac{145.5}{6.7} = 21.71\ \frac{\mathrm{KN}}{\mathrm{m}}$$

$$21.71\ \times \frac{2.2^2}{2} = 52.54\ \mathrm{KN.m}$$

Live load:

$(6.85 \times 1.6 + 3.5 \times 1.2) \times 5 = 63.25\ \mathrm{KN}$

$$\frac{63.25}{6.15} = 10.28\ \frac{\text{KN}}{\text{m}}$$

Live Moment:

$$\frac{63.25}{6.7} = 9.44\ \frac{\text{KN}}{\text{m}}$$

$$9.44\ \times \frac{2.2^2}{2} = 22.84\ \text{KN.m}$$

6-4-1-4) Third Floor:

A) Dead Loads:

P1

$$\text{Area of Isolation: } 2.95 \times 3.35 + 2.5 \times 2.1 = 15.13\ \text{m}^2$$

$$7.83 + \frac{1 \times 15.13}{36.725} = 8.24\ \frac{\text{KN}}{\text{m}^2}$$

P2

$$\text{Area of Isolation: } 0.95 \times 3.45 + 6.05 \times 1.1 = 9.93\ \text{m}^2$$

$$7.83 + \frac{1 \times 9.93}{27.97} = 8.19\ \frac{\text{KN}}{\text{m}^2}$$

P3

$$\frac{102.22}{10.68} = 9.57\ \frac{\text{KN}}{\text{m}^2}$$

P4

$$7.83 + 1 = 8.83\ \frac{\text{KN}}{\text{m}^2}$$

P5

$$7.83 + 1 = 8.83\ \frac{\mathrm{KN}}{\mathrm{m}^2}$$

P6,P7

$$8.88\ \frac{\mathrm{KN}}{\mathrm{m}^2}$$

Equivalent dead load of the partitions

$$15 \times 2.6 \times 3.6 + 5 \times 2.6 \times 2.1 = 167.7\ \mathrm{KN}$$

$$\frac{167.7}{115.78} = 1.45\ \frac{\mathrm{KN}}{\mathrm{m}^2}$$

Walls of the perimeter

Inside: $3.6 \times (3 - 0.5) = 9\ \frac{\mathrm{KN}}{\mathrm{m}}$

Outside: $1 \times 3.6 = 3.6\ \frac{\mathrm{KN}}{\mathrm{m}}$

B) Live Loads:

$$\text{Total of the plan: } 2\ \frac{\mathrm{KN}}{\mathrm{m}^2}$$

$$\text{Stairs: } 3.5\ \frac{\mathrm{KN}}{\mathrm{m}^2}$$

C) Balconies:

Lower balcony:

Dead load:

$$\text{Diaphragm}: 4.5 \times 1 \ \times 7.83 = 35.24 \text{ KN}$$

$$\frac{35.24}{5.9} = 5.97 \ \frac{\text{KN}}{\text{m}}$$

Dead Moment:

$$\frac{35.24}{4.5} = 7.83 \ \frac{\text{KN}}{\text{m}}$$

$$7.83 \ \times \frac{1^2}{2} = 3.92 \text{ KN.m}$$

Live load:

$$4.5 \times 1 \times 5 = 22.5 \text{ KN}$$

$$\frac{22.5}{5.9} = 3.81 \ \frac{\text{KN}}{\text{m}}$$

Live Moment:

$$\frac{22.5}{4.5} = 5 \ \frac{\text{KN}}{\text{m}}$$

$$5 \ \times \frac{1^2}{2} = 2.5 \text{ KN.m}$$

Upper balcony:

Dead load:

$$\text{Diaphragm}: 1.6 \times 6.6 \times 7.83 = 82.68 \text{ KN}$$

$$1.6 \times 6.6 \times 1 = 10.56 \text{ KN}$$

$$\text{Total}: 82.68 + 10.56 = 93.24 \text{ KN}$$

$$\frac{93.24}{6.15} = 15.16 \ \frac{\text{KN}}{\text{m}}$$

Dead Moment:

$$\frac{93.24}{6.6} = 14.13\ \frac{\text{KN}}{\text{m}}$$

$$14.13\ \times \frac{1.6^2}{2} = 18.09\ \text{KN.m}$$

Live load:

$$1.6 \times 6.6 \times 5 = 52.8\ \text{KN}$$

$$\frac{52.8}{6.15} = 8.59\ \frac{\text{KN}}{\text{m}}$$

Live Moment:

$$\frac{52.8}{6.6} = 8\ \frac{\text{KN}}{\text{m}}$$

$$8\ \times \frac{1.6^2}{2} = 10.24\ \text{KN.m}$$

6-4-1-5) Fourth & Fifth Floors:

A) Dead Loads:

P1

$$\text{Area of Isolation: } 2.95 \times 3.35 + 2.5 \times 2.1 = 15.13\ \text{m}^2$$

$$7.83 + \frac{1 \times 15.13}{36.725} = 8.24\ \frac{\text{KN}}{\text{m}^2}$$

P2

$$\text{Area of Isolation: } 0.95 \times 3.45 + 6.05 \times 1.1 = 9.93\ \text{m}^2$$

$$7.85 + \frac{1 \times 9.93}{27.97} = 8.19\ \frac{\text{KN}}{\text{m}^2}$$

$\boxed{\textbf{P3}}$

$$\frac{102.22}{10.68} = 9.57\ \frac{\mathrm{KN}}{\mathrm{m}^2}$$

$\boxed{\textbf{P6,P7}}$

$$8.88\ \frac{\mathrm{KN}}{\mathrm{m}^2}$$

$\boxed{\textbf{Equivalent dead load of the partitions}}$

$$15 \times 2.6 \times 3.6 + 5 \times 2.6 \times 2.1 = 167.7\ \mathrm{KN}$$

$$\frac{167.7}{115.78} = 1.45\ \frac{\mathrm{KN}}{\mathrm{m}^2}$$

$\boxed{\textbf{Walls of the perimeter}}$

Inside: $3.6 \times (3 - 0.5) = 9\ \frac{\mathrm{KN}}{\mathrm{m}}$

B) Live Loads:

$$\text{Total of the plan: } 2\ \frac{\mathrm{KN}}{\mathrm{m}^2}$$

$$\text{Stairs: } 3.5\ \frac{\mathrm{KN}}{\mathrm{m}^2}$$

C) Balconies:

Lower balcony:

Dead load:

Left Beam:

$$\text{Diaphragm: } 4.5 \times 2.5 \times 7.83 = 88.09 \text{ KN}$$

$$4.5 \times 2.5 \times 1 = 11.25 \text{ KN}$$

$$\text{Total: } 88.09 + 11.25 = 99.34 \text{ KN}$$

$$\frac{99.34}{5.9} = 16.84 \frac{\text{KN}}{\text{m}}$$

Right Beam:

$$\text{Diaphragm: } (8.3 - 4.5) \times 2.5 \times 7.83 = 74.39 \text{ KN}$$

$$(8.3 - 4.5) \times 2.5 \times 1 = 9.5 \text{ KN}$$

$$\text{Total: } 74.39 + 9.5 = 83.89 \text{ KN}$$

$$\frac{83.89}{2.9} = 28.93 \frac{\text{KN}}{\text{m}}$$

Dead Moment:

Left Beam:

$$\frac{99.34}{5.9} = 16.84 \frac{\text{KN}}{\text{m}}$$

$$16.84 \times \frac{2.5^2}{2} = 52.63 \text{ KN.m}$$

Right Beam:

$$\frac{83.89}{2.9} = 28.93 \frac{\text{KN}}{\text{m}}$$

$$28.93 \times \frac{2.5^2}{2} = 90.41 \text{ KN.m}$$

Live load:

Left Beam:

$$4.5 \times 2.5 \times 5 = 56.25 \text{ KN}$$

$$\frac{56.25}{5.9} = 9.53\ \frac{\text{KN}}{\text{m}}$$

Right Beam:

$$(8.3 - 4.5) \times 2.5 \times 5 = 47.5\ \text{KN}$$

$$\frac{47.5}{2.9} = 16.38\ \frac{\text{KN}}{\text{m}}$$

Live Moment:

Left Beam:

$$\frac{56.25}{5.9} = 9.53\ \frac{\text{KN}}{\text{m}}$$

$$9.53\ \times \frac{2.5^2}{2} = 29.78\ \text{KN.m}$$

Right Beam:

$$\frac{47.5}{2.9} = 16.38\ \frac{\text{KN}}{\text{m}}$$

$$16.38\ \times \frac{2.5^2}{2} = 51.19\ \text{KN.m}$$

Upper balcony:

Dead load:

$$\text{Diaphragm: } 5.1 \times 1.6 \times 7.83 = 63.89\ \text{KN}$$

$$5.1 \times 1.6 \times 1 = 8.16\ \text{KN}$$

$$\text{Total: } 63.89 + 8.16 = 72.05\ \text{KN}$$

$$\frac{72.05}{6.15} = 11.71\ \frac{\text{KN}}{\text{m}}$$

Dead Moment:

$$\frac{72.05}{6.15} = 11.71\ \frac{\mathrm{KN}}{\mathrm{m}}$$

$$11.71\ \times \frac{1.6^2}{2} = 14.99\ \mathrm{KN.m}$$

Live load:

$$5.1 \times 1.6 \times 5 = 40.8\ \mathrm{KN}$$

$$\frac{40.8}{6.15} = 6.63\ \frac{\mathrm{KN}}{\mathrm{m}}$$

Live Moment:

$$\frac{40.8}{6.15} = 6.63\ \frac{\mathrm{KN}}{\mathrm{m}}$$

$$6.63\ \times \frac{1.6^2}{2} = 8.49\ \mathrm{KN.m}$$

6-4-1-6) Sixth Floor (Roof):

A) Dead Loads:

P1

$$7.87 + 2.5 = 10.37\ \frac{\mathrm{KN}}{\mathrm{m^2}}$$

P2

$$7.83 + 2.5 = 10.33\ \frac{\mathrm{KN}}{\mathrm{m^2}}$$

P3

$$7.87 + 2.5 = 10.37\ \frac{\text{KN}}{\text{m}^2}$$

P6,P7

$$8.88\ \frac{\text{KN}}{\text{m}^2}$$

Equivalent dead load of the partitions

$$5 \times 1 \times 3.6 = 18\ \text{KN}$$

$$\text{Area: } 9.8 \times 11 - 1.2 \times 1.9 - 5.95 \times 3.3 = 85.89\ \text{m}^2$$

$$\frac{18}{85.89} = 0.21\ \frac{\text{KN}}{\text{m}^2}$$

Walls of the perimeter

$$3.6 \times 1 = 3.6\ \frac{\text{KN}}{\text{m}}$$

Walls of the around stairs

$$2.2 \times 3.6 = 7.92\ \frac{\text{KN}}{\text{m}}$$

B) Live Loads:

$$\text{Total of the plan: } 2\ \frac{\text{KN}}{\text{m}^2}$$

$$\text{Stairs: } 3.5\ \frac{\text{KN}}{\text{m}^2}$$

C) Balconies:

Lower balcony:

Dead load:

Left Beam:

$$\text{Diaphragm: } 7.85 \times 6.5 \times 1 = 51.03 \text{ KN}$$

$$\frac{51.03}{5.9} = 8.65 \ \frac{\text{KN}}{\text{m}}$$

Right Beam:

$$\text{Diaphragm: } 7.85 \times 3.3 \times 1 = 25.91 \text{ KN}$$

$$\frac{25.91}{2.9} = 8.93 \ \frac{\text{KN}}{\text{m}}$$

Dead Moment:

Left Beam:

$$\frac{51.03}{5.9} = 8.65 \ \frac{\text{KN}}{\text{m}}$$

$$8.65 \ \times \frac{1^2}{2} = 4.33 \text{ KN.m}$$

Right Beam:

$$\frac{25.91}{2.9} = 8.93 \ \frac{\text{KN}}{\text{m}}$$

$$8.93 \ \times \frac{1^2}{2} = 4.47 \text{ KN.m}$$

Live load:

Left Beam:

$$6.5 \times 1 \times 5 = 32.5 \text{ KN}$$

$$\frac{32.5}{5.9} = 5.51\ \frac{\text{KN}}{\text{m}}$$

Right Beam:

$$3.3 \times 1 \times 5 = 16.5\ \text{KN}$$

$$\frac{16.5}{2.9} = 5.69\ \frac{\text{KN}}{\text{m}}$$

Live Moment:

Left Beam:

$$\frac{32.5}{5.9} = 5.51\ \frac{\text{KN}}{\text{m}}$$

$$5.51\ \times \frac{1^2}{2} = 2.76\ \text{KN.m}$$

Right Beam:

$$\frac{16.5}{2.9} = 5.69\ \frac{\text{KN}}{\text{m}}$$

$$5.69\ \times \frac{1^2}{2} = 2.85\ \text{KN.m}$$

Upper balcony:

Dead load:

$$7.87 \times 1.35 \times 5.1 = 54.18\ \text{KN}$$

$$\frac{54.18}{6.2} = 8.74\ \frac{\text{KN}}{\text{m}}$$

Dead Moment:

$$\frac{54.18}{6.2} = 8.74\ \frac{\text{KN}}{\text{m}}$$

$$8.74 \times \frac{1.35^2}{2} = 7.96 \text{ KN.m}$$

Snow load:

$$\text{Total of the plan: } 0.75 \ \frac{\text{KN}}{\text{m}^2}$$

6-4-1-7) Seventh Floor (Lift Room Roof):

A) Dead Loads:

$$7.83 + 1 = 8.83 \ \frac{\text{KN}}{\text{m}^2}$$

B) Live Loads:

$$\text{Total of the plan: } 1 \ \frac{\text{KN}}{\text{m}^2}$$

C) Snow Loads:

$$\text{Total of the plan: } 0.75 \ \frac{\text{KN}}{\text{m}^2}$$

6-4-2) Loading Areas divisions:

Based on the drawings, loading areas are divided and based on this we can calculate the amount of the loads for each beam in each storey:

A) Underground:

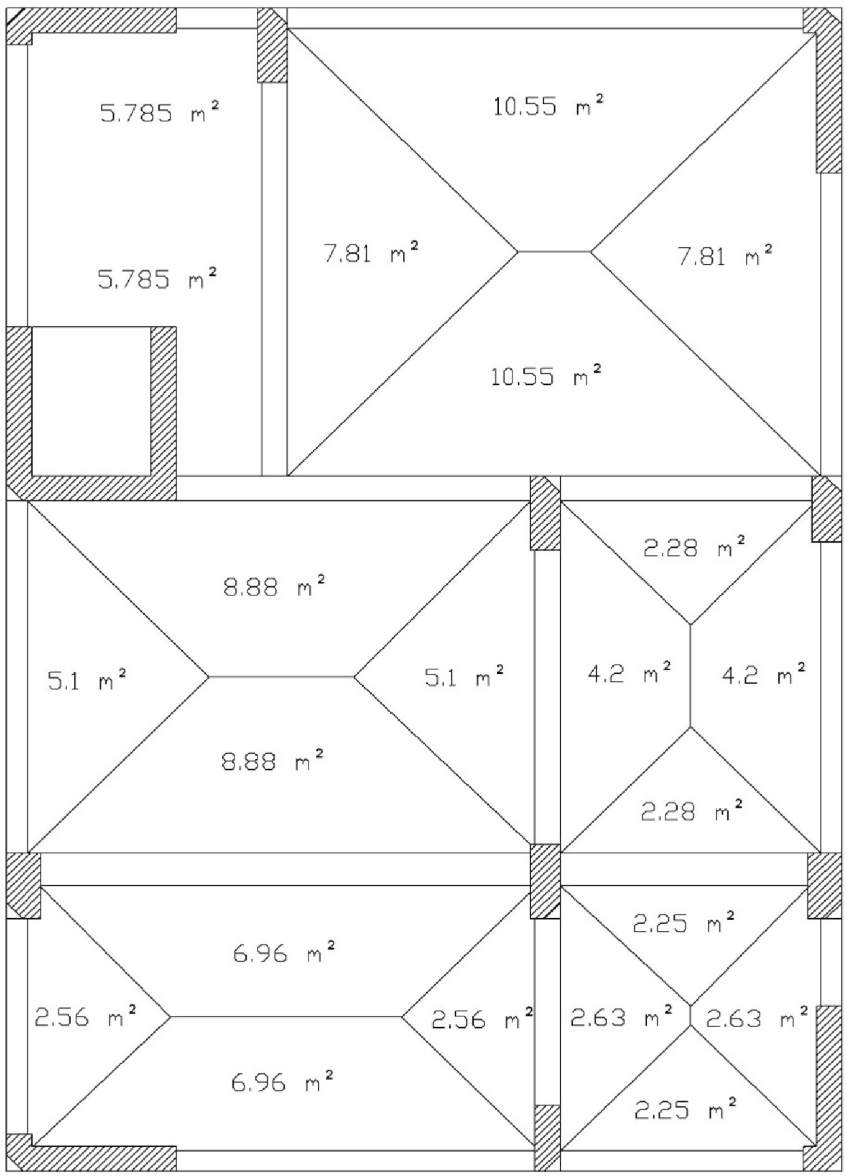

Figure 6-1- Loading Areas divisions- Part 1

B) Ground Floor:

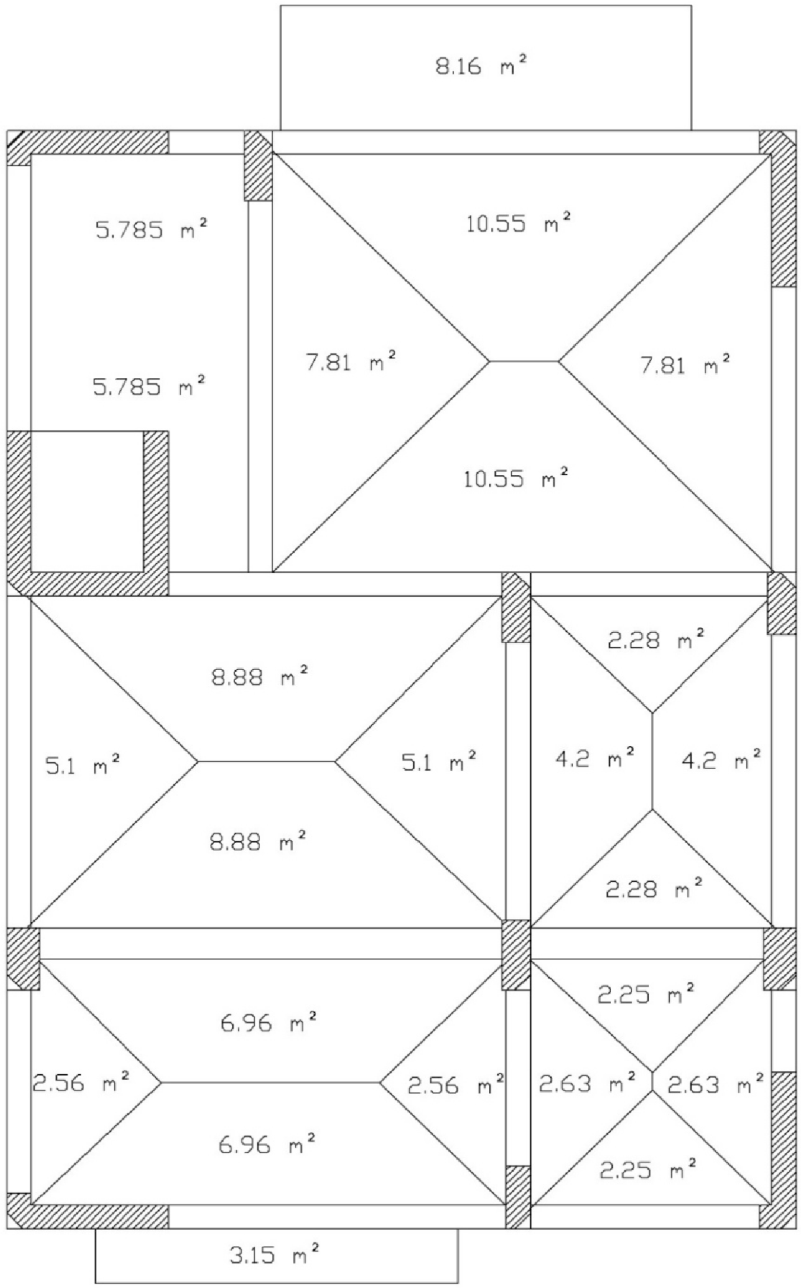

Figure 6-2- Loading Areas divisions- Part 2

C) First Floor:

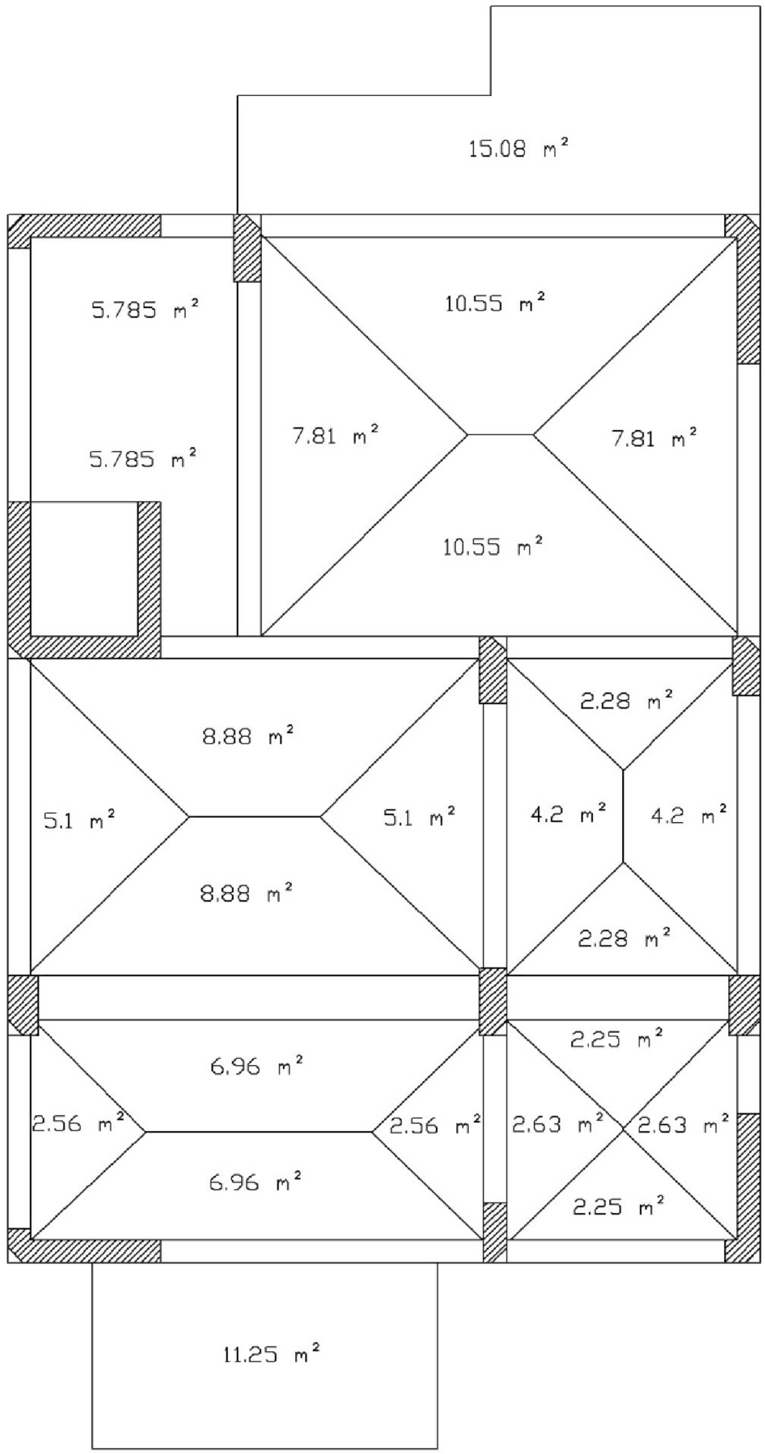

Figure 6-3- Loading Areas divisions- Part 3

D) Second Floor:

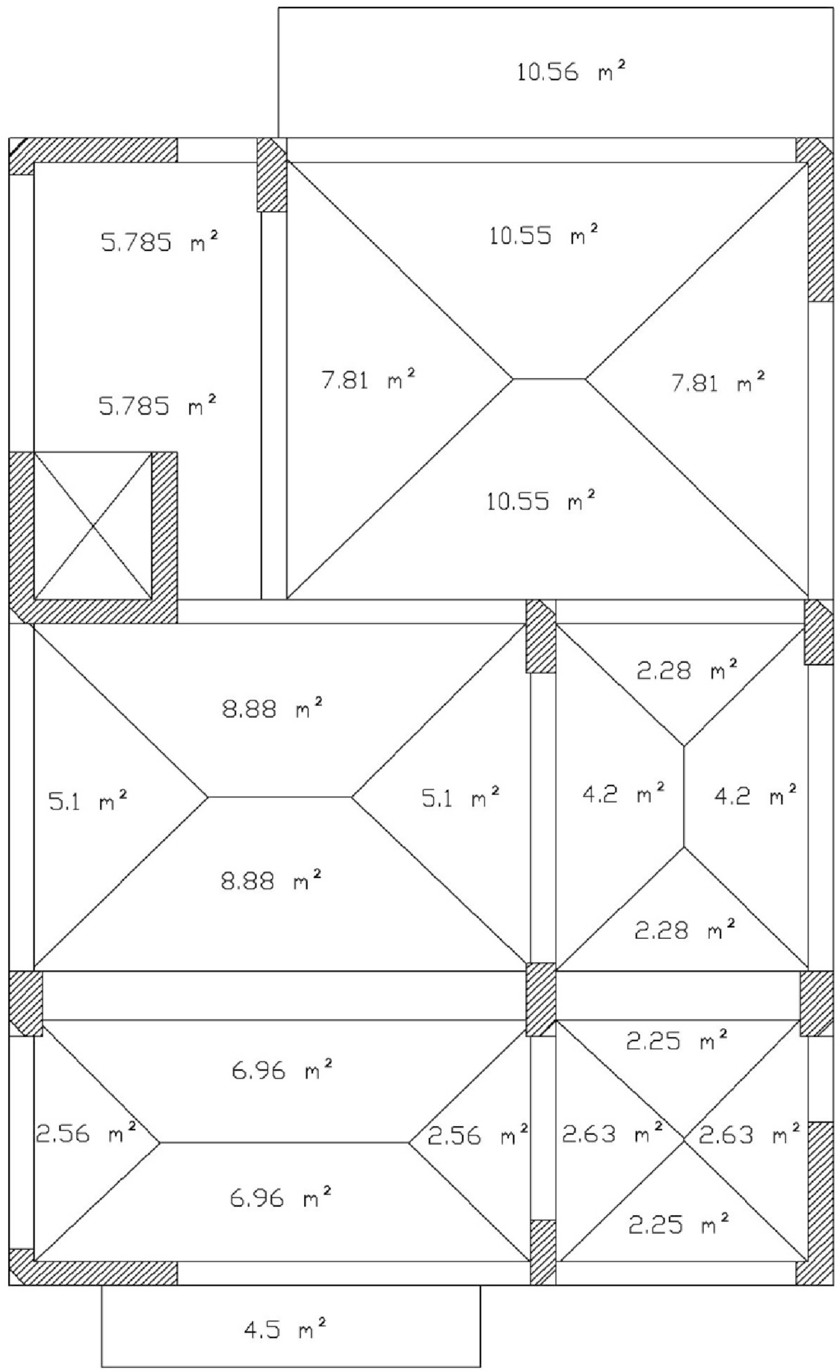

Figure 6-4- Loading Areas divisions- Part 4

E) Third Floor:

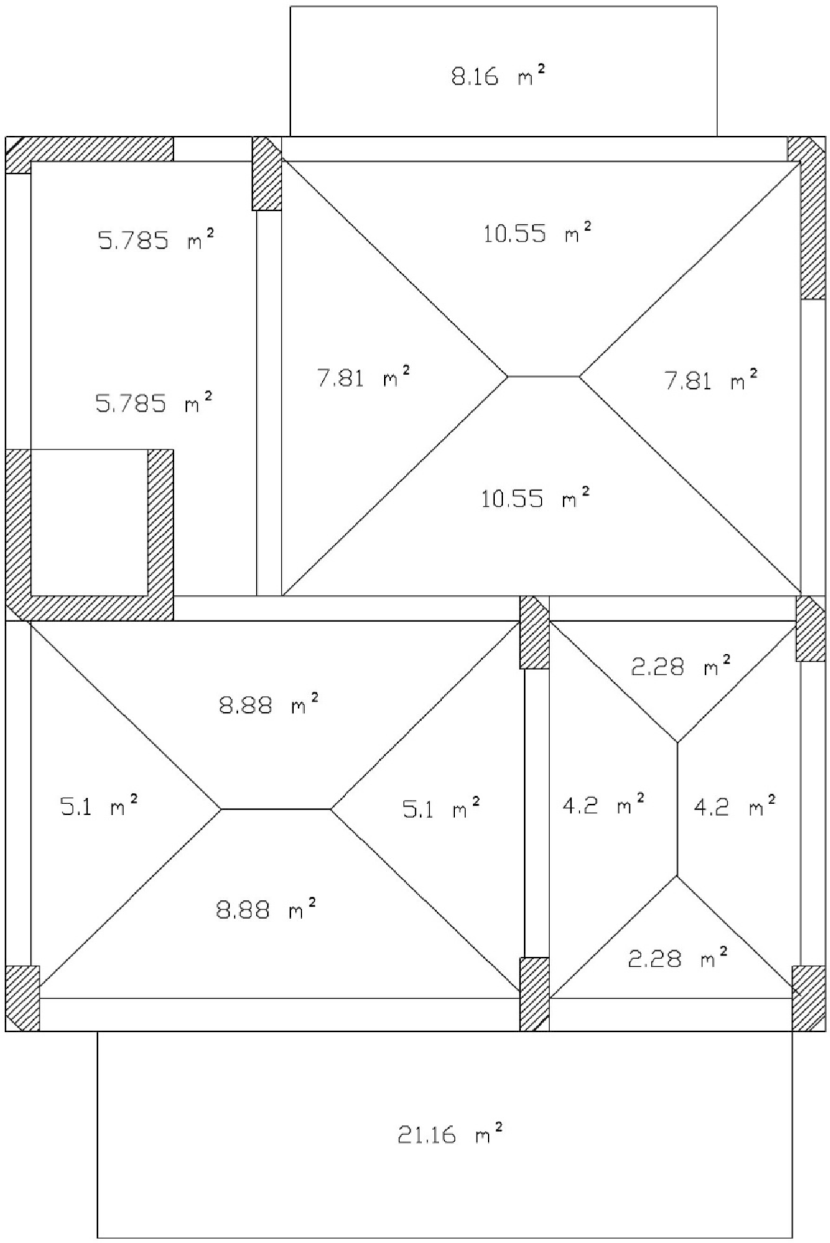

Figure 6-5- Loading Areas divisions- Part 5

F) Fourth Floor:

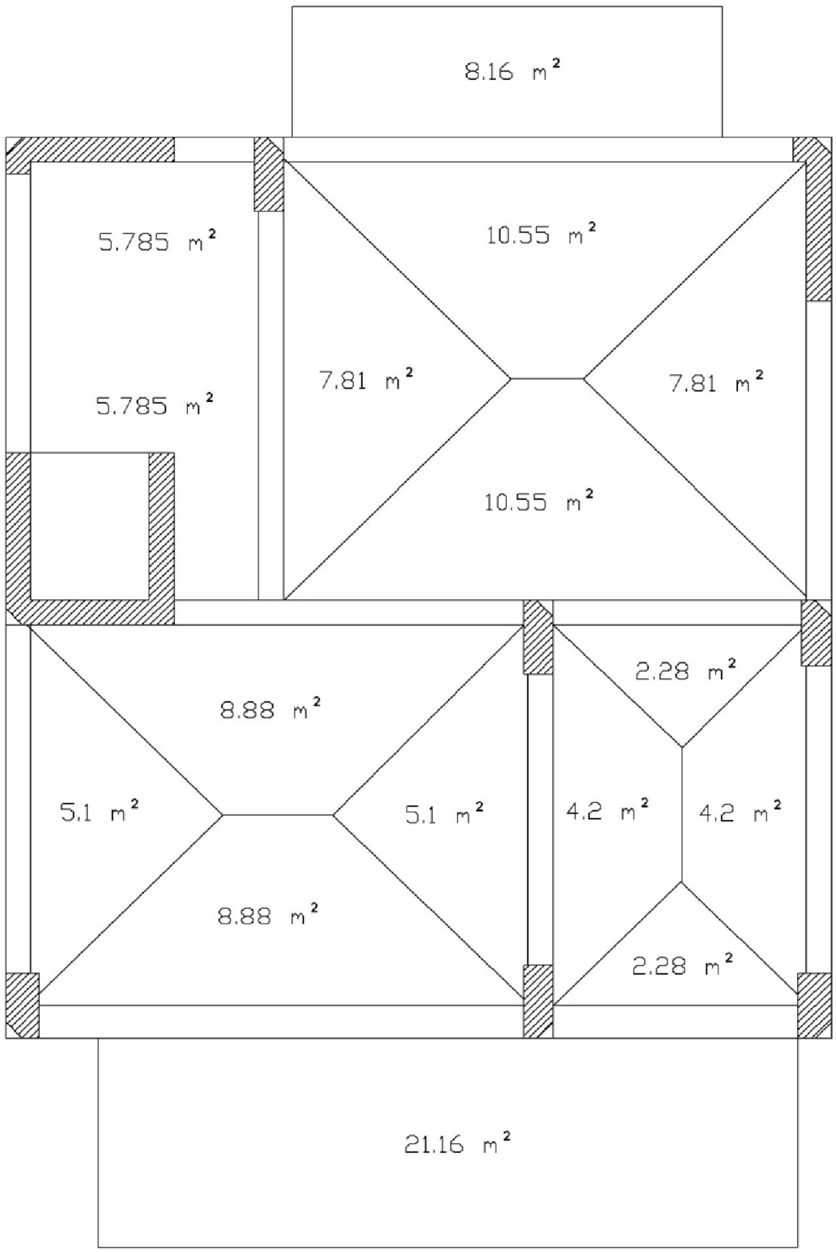

Figure 6-6- Loading Areas divisions- Part 6

G) Fifth Floor:

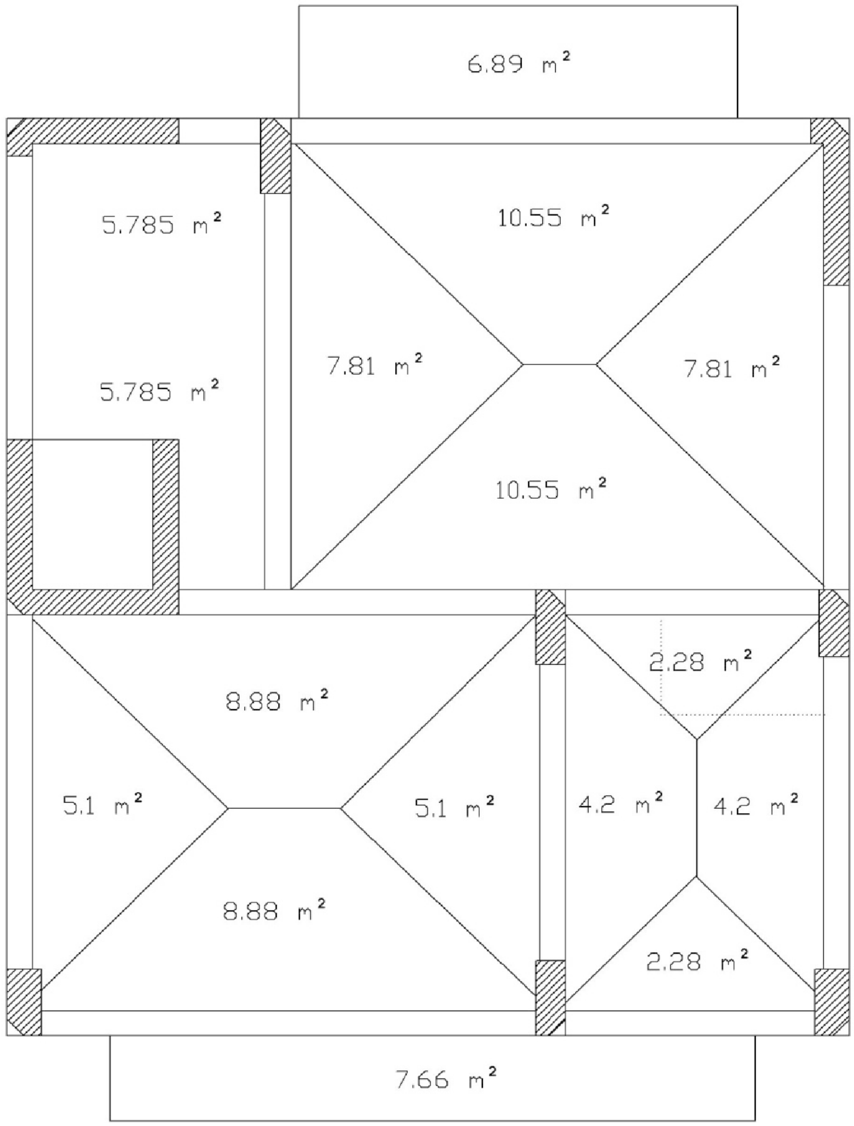

Figure 6-7- Loading Areas divisions- Part 7

H) Sixth (Lift Room) Floor:

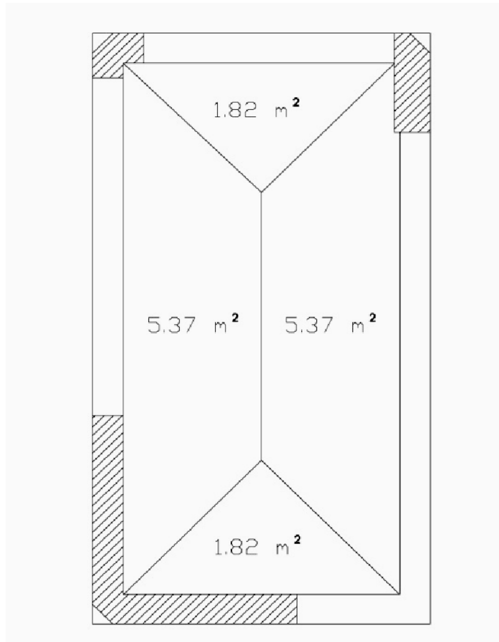

Figure 6-8- Loading Areas divisions- Part 8

6-4-3) Beams' Numbers:

For simplicity reasons, we defined a number to each beam and we will use these numbers for calculation of the loads for the beams. These numbers are shown in below:

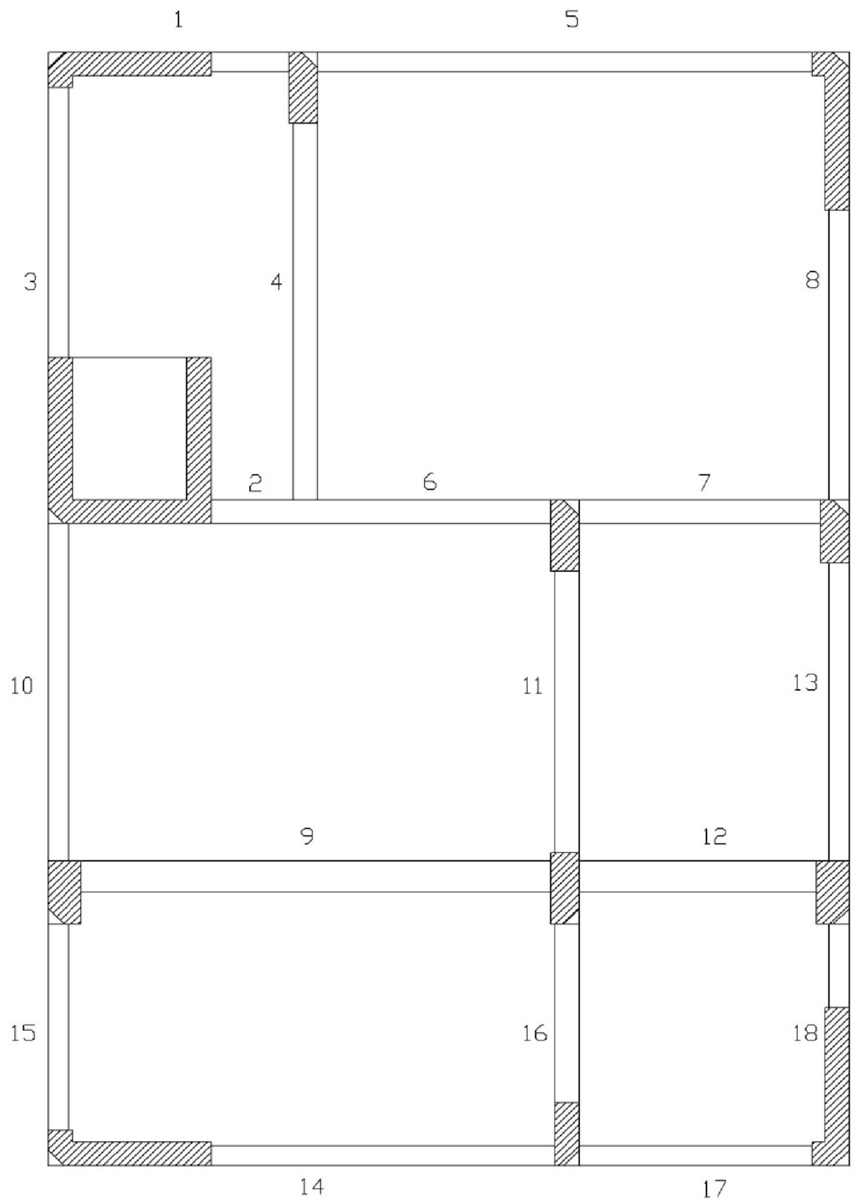

Figure 6-9- Beams' Numbers

6-4-4) Distribution of the loads (Beam's Loading calculations):

A) Ground Floor:

Ground Floor(z=+0.00m)

Primary									
Area Name	**Beam**	**Beam Length (m)**	**Related Area (m^2)**	**Dead (KN/m^2)**	**Partitions (KN/m^2)**	**Total Dead (KN/m^2)**	**Live (KN/m^2)**	**Total Dead (KN/m)**	**Live (KN/m)**
P1	**4**	5.65	7.81	7.87	1.03	8.9	5	12.30	6.91
	5	6.5	10.55	7.87	1.03	8.9	5	14.45	8.12
	6	3.25	5.275	7.87	1.03	8.9	5	14.45	8.12
	7	3.25	5.275	7.87	1.03	8.9	5	14.45	8.12
	8	5.65	7.81	7.87	1.03	8.9	5	12.30	6.91
P2	**2**	3.25	4.44	7.87	1.03	8.9	5	12.16	6.83
	6	3.25	4.44	7.87	1.03	8.9	5	12.16	6.83
	9	6.5	8.88	7.87	1.03	8.9	5	12.16	6.83
	10	5.75	5.1	7.87	1.03	8.9	5	7.89	4.43
	11	5.75	5.1	7.87	1.03	8.9	5	7.89	4.43
P3	**7**	3.25	2.28	7.87	1.03	8.9	5	6.24	3.51
	12	3.25	2.28	7.87	1.03	8.9	5	6.24	3.51
	11	5.75	4.2	7.87	1.03	8.9	5	6.50	3.65
	13	5.75	4.2	7.87	1.03	8.9	5	6.50	3.65
P4	**9**	6.5	6.96	7.87	1.03	8.9	5	9.53	5.35
	14	6.5	6.96	7.87	1.03	8.9	5	9.53	5.35
	15	3.45	2.56	7.87	1.03	8.9	5	6.60	3.71
	16	3.45	2.56	7.87	1.03	8.9	5	6.60	3.71
P5	**12**	3.25	2.25	7.87	1.03	8.9	5	6.16	3.46
	17	3.25	2.25	7.87	1.03	8.9	5	6.16	3.46
	16	3.45	2.63	7.87	1.03	8.9	5	6.78	3.81
	18	3.45	2.63	7.87	1.03	8.9	5	6.78	3.81
P6,P7	**1**	3.25	5.785	8.88		8.88	3.5	15.81	6.23
	2	3.25	5.785	8.88		8.88	3.5	15.81	6.23
	3	5.65				0	3.5	0.00	0.00
	4	5.65				0	3.5	0.00	0.00

Table 6-8- Distribution of the loads- Part 1

Final			
Beam	**Total Dead (KN/m)**	**Live (KN/m)**	**Primeter wall (KN/m)**
1	15.81	6.23	8.28
2	27.97	13.06	-
3	0.00	0.00	8.28
4	12.30	6.91	-
5	14.45	8.12	8.28
6	26.60	14.95	-
7	20.69	11.62	-
8	12.30	6.91	8.28
9	21.69	12.18	-
10	7.89	4.43	8.28
11	14.39	8.09	-
12	12.41	6.97	-
13	6.50	3.65	8.28
14	9.53	5.35	8.28
15	6.60	3.71	8.28
16	13.39	7.52	-
17	6.16	3.46	8.28
18	6.78	3.81	8.28

Table 6-9- Distribution of the loads- Part 2

B) First Floor:

1st Floor(z=+2.80m)

Primary									
Area Name	Beam	Beam Length (m)	Related Area (m^2)	Dead (KN/m^2)	Partitions (KN/m^2)	Total Dead (KN/m^2)	Live (KN/m^2)	Total Dead (KN/m)	Live (KN/m)
P1	4	5.65	7.81	8.23	2.16	10.39	2	14.36	2.76
	5	6.5	10.55	8.23	2.16	10.39	2	16.86	3.25
	6	3.25	5.275	8.23	2.16	10.39	2	16.86	3.25
	7	3.25	5.275	8.23	2.16	10.39	2	16.86	3.25
	8	5.65	7.81	8.23	2.16	10.39	2	14.36	2.76
P2	2	3.25	4.44	7.83	2.16	9.99	2	13.65	2.73
	6	3.25	4.44	7.83	2.16	9.99	2	13.65	2.73
	9	6.5	8.88	7.83	2.16	9.99	2	13.65	2.73
	10	5.75	5.1	7.83	2.16	9.99	2	8.86	1.77
	11	5.75	5.1	7.83	2.16	9.99	2	8.86	1.77
P3	7	3.25	2.28	8.12	2.16	10.28	2	7.21	1.40
	12	3.25	2.28	8.12	2.16	10.28	2	7.21	1.40
	11	5.75	4.2	8.12	2.16	10.28	2	7.51	1.46
	13	5.75	4.2	8.12	2.16	10.28	2	7.51	1.46
P4	9	6.5	6.96	8.44	2.16	10.6	2	11.35	2.14
	14	6.5	6.96	8.44	2.16	10.6	2	11.35	2.14
	15	3.45	2.56	8.44	2.16	10.6	2	7.87	1.48
	16	3.45	2.56	8.44	2.16	10.6	2	7.87	1.48
P5	12	3.25	2.25	8.44	2.16	10.6	2	7.34	1.38
	17	3.25	2.25	8.44	2.16	10.6	2	7.34	1.38
	16	3.45	2.63	8.44	2.16	10.6	2	8.08	1.52
	18	3.45	2.63	8.44	2.16	10.6	2	8.08	1.52
P6,P7	1	3.25	5.785	8.88		8.88	3.5	15.81	6.23
	2	3.25	5.785	8.88		8.88	3.5	15.81	6.23
	3	5.65				0	3.5	0.00	0.00
	4	5.65				0	3.5	0.00	0.00

Table 6-10- Distribution of the loads- Part 3

Final			
Beam	**Total Dead (KN/m)**	**Live (KN/m)**	**Primeter wall (KN/m)**
1	15.81	6.23	9.00
2	29.45	8.96	-
3	0.00	0.00	9.00
4	14.36	2.76	-
5	31.42	9.88	
6	30.51	5.98	-
7	24.08	4.65	-
8	14.36	2.76	9.00
9	25.00	4.87	-
10	8.86	1.77	9.00
11	16.37	3.23	-
12	14.55	2.79	-
13	7.51	1.46	9.00
14	16.64	4.69	
15	7.87	1.48	9.00
16	15.95	3.01	-
17	7.34	1.38	9.00
18	8.08	1.52	9.00

Table 6-11- Distribution of the loads- Part 4

Balcony				
Beam	**Dead (KN/m)**	**Live (KN/m)**	**Dead Moment(KN.m)**	**Live Moment(KN.m)**
5	14.56	6.63	22.46	10.24
14	5.29	2.55	1.78	0.86

Table 6-12- Distribution of the loads- Part 5

C) Second Floor:

2nd Floor(z=+5.80m)

Primary									
Area Name	**Beam**	**Beam Length (m)**	**Related Area (m^2)**	**Dead (KN/m^2)**	**Partitions (KN/m^2)**	**Total Dead (KN/m^2)**	**Live (KN/m^2)**	**Total Dead (KN/m)**	**Live (KN/m)**
P1	4	5.65	7.81	8.23	1.8	10.03	2	13.86	2.76
	5	6.5	10.55	8.23	1.8	10.03	2	16.28	3.25
	6	3.25	5.275	8.23	1.8	10.03	2	16.28	3.25
	7	3.25	5.275	8.23	1.8	10.03	2	16.28	3.25
	8	5.65	7.81	8.23	1.8	10.03	2	13.86	2.76
P2	2	3.25	4.44	7.83	1.8	9.63	2	13.16	2.73
	6	3.25	4.44	7.83	1.8	9.63	2	13.16	2.73
	9	6.5	8.88	7.83	1.8	9.63	2	13.16	2.73
	10	5.75	5.1	7.83	1.8	9.63	2	8.54	1.77
	11	5.75	5.1	7.83	1.8	9.63	2	8.54	1.77
P3	7	3.25	2.28	9.57	1.8	11.37	2	7.98	1.40
	12	3.25	2.28	9.57	1.8	11.37	2	7.98	1.40
	11	5.75	4.2	9.57	1.8	11.37	2	8.31	1.46
	13	5.75	4.2	9.57	1.8	11.37	2	8.31	1.46
P4	9	6.5	6.96	7.85	1.8	9.65	2	10.33	2.14
	14	6.5	6.96	7.85	1.8	9.65	2	10.33	2.14
	15	3.45	2.56	7.85	1.8	9.65	2	7.16	1.48
	16	3.45	2.56	7.85	1.8	9.65	2	7.16	1.48
P5	12	3.25	2.25	8.41	1.8	10.21	2	7.07	1.38
	17	3.25	2.25	8.41	1.8	10.21	2	7.07	1.38
	16	3.45	2.63	8.41	1.8	10.21	2	7.78	1.52
	18	3.45	2.63	8.41	1.8	10.21	2	7.78	1.52
P6,P7	1	3.25	5.785	8.88		8.88	3.5	15.81	6.23
	2	3.25	5.785	8.88		8.88	3.5	15.81	6.23
	3	5.65				0	3.5	0.00	0.00
	4	5.65				0	3.5	0.00	0.00

Table 6-13- Distribution of the loads- Part 6

Final			
Beam	Total Dead (KN/m)	Live (KN/m)	Primeter wall (KN/m)
1	15.81	6.23	9.00
2	28.96	8.96	-
3	0.00	0.00	9.00
4	13.86	2.76	-
5	39.94	13.53	
6	29.44	5.98	-
7	24.26	4.65	-
8	13.86	2.76	9.00
9	23.49	4.87	-
10	8.54	1.77	9.00
11	16.85	3.23	-
12	15.04	2.79	-
13	8.31	1.46	9.00
14	27.17	11.67	
15	7.16	1.48	9.00
16	14.94	3.01	-
17	7.07	1.38	9.00
18	7.78	1.52	9.00

Table 6-14- Distribution of the loads- Part 7

Balcony				
Beam	Dead (KN/m)	Live (KN/m)	Dead Moment(KN.m)	Live Moment(KN.m)
5	23.66	10.28	52.54	22.84
14	16.84	9.53	68.97	39.06

Table 6-15- Distribution of the loads- Part 8

D) Third Floor:

3rd Floor(z=+8.80m)

Primary									
Area Name	**Beam**	**Beam Length (m)**	**Related Area (m^2)**	**Dead (KN/m^2)**	**Partitions (KN/m^2)**	**Total Dead (KN/m^2)**	**Live (KN/m^2)**	**Total Dead (KN/m)**	**Live (KN/m)**
P1	**4**	5.65	7.81	8.24	1.45	9.69	2	13.39	2.76
	5	6.5	10.55	8.24	1.45	9.69	2	15.73	3.25
	6	3.25	5.275	8.24	1.45	9.69	2	15.73	3.25
	7	3.25	5.275	8.24	1.45	9.69	2	15.73	3.25
	8	5.65	7.81	8.24	1.45	9.69	2	13.39	2.76
P2	**2**	3.25	4.44	8.19	1.45	9.64	2	13.17	2.73
	6	3.25	4.44	8.19	1.45	9.64	2	13.17	2.73
	9	6.5	8.88	8.19	1.45	9.64	2	13.17	2.73
	10	5.75	5.1	8.19	1.45	9.64	2	8.55	1.77
	11	5.75	5.1	8.19	1.45	9.64	2	8.55	1.77
P3	**7**	3.25	2.28	9.57	1.45	11.02	2	7.73	1.40
	12	3.25	2.28	9.57	1.45	11.02	2	7.73	1.40
	11	5.75	4.2	9.57	1.45	11.02	2	8.05	1.46
	13	5.75	4.2	9.57	1.45	11.02	2	8.05	1.46
P4	**9**	6.5	6.96	8.83	1.45	10.28	2	11.01	2.14
	14	6.5	6.96	8.83	1.45	10.28	2	11.01	2.14
	15	3.45	2.56	8.83	1.45	10.28	2	7.63	1.48
	16	3.45	2.56	8.83	1.45	10.28	2	7.63	1.48
P5	**12**	3.25	2.25	8.83	1.45	10.28	2	7.12	1.38
	17	3.25	2.25	8.83	1.45	10.28	2	7.12	1.38
	16	3.45	2.63	8.83	1.45	10.28	2	7.84	1.52
	18	3.45	2.63	8.83	1.45	10.28	2	7.84	1.52
P6,P7	**1**	3.25	5.785	8.88		8.88	3.5	15.81	6.23
	2	3.25	5.785	8.88		8.88	3.5	15.81	6.23
	3	5.65				0	3.5	0.00	0.00
	4	5.65				0	3.5	0.00	0.00

Table 6-16- Distribution of the loads- Part 9

Final			
Beam	**Total Dead (KN/m)**	**Live (KN/m)**	**Primeter wall (KN/m)**
1	15.81	6.23	9.00
2	28.98	8.96	-
3	0.00	0.00	9.00
4	13.39	2.76	-
5	30.89	11.84	
6	28.90	5.98	-
7	23.46	4.65	-
8	13.39	2.76	9.00
9	24.18	4.87	-
10	8.55	1.77	9.00
11	16.60	3.23	-
12	14.85	2.79	-
13	8.05	1.46	9.00
14	16.98	5.95	3.60
15	7.63	1.48	3.60
16	15.46	3.01	-
17	7.12	1.38	3.60
18	7.84	1.52	3.60

Table 6-17- Distribution of the loads- Part 10

Balcony				
Beam	**Dead (KN/m)**	**Live (KN/m)**	**Dead Moment(K N.m)**	**Live Moment(K N.m)**
5	15.16	8.59	3.92	2.50
14	5.97	3.81	18.09	10.24

Table 6-18- Distribution of the loads- Part 11

E) Fourth & Fifth Floors:

4th & 5th Floors(z=+11.80m,z=+14.80m)

Primary									
Area Name	Beam	Beam Length (m)	Related Area (m^2)	Dead (KN/m^2)	Partitions (KN/m^2)	Total Dead (KN/m^2)	Live (KN/m^2)	Total Dead (KN/m)	Live (KN/m)
P1	4	5.65	7.81	8.24	1.45	9.69	2	13.39	2.76
	5	6.5	10.55	8.24	1.45	9.69	2	15.73	3.25
	6	3.25	5.275	8.24	1.45	9.69	2	15.73	3.25
	7	3.25	5.275	8.24	1.45	9.69	2	15.73	3.25
	8	5.65	7.81	8.24	1.45	9.69	2	13.39	2.76
P2	2	3.25	4.44	8.19	1.45	9.64	2	13.17	2.73
	6	3.25	4.44	8.19	1.45	9.64	2	13.17	2.73
	9	6.5	8.88	8.19	1.45	9.64	2	13.17	2.73
	10	5.75	5.1	8.19	1.45	9.64	2	8.55	1.77
	11	5.75	5.1	8.19	1.45	9.64	2	8.55	1.77
P3	7	3.25	2.28	9.57	1.45	11.02	2	7.73	1.40
	12	3.25	2.28	9.57	1.45	11.02	2	7.73	1.40
	11	5.75	4.2	9.57	1.45	11.02	2	8.05	1.46
	13	5.75	4.2	9.57	1.45	11.02	2	8.05	1.46
P4	9	6.5	6.96			0		0.00	0.00
	14	6.5	6.96			0		0.00	0.00
	15	3.45	2.56			0		0.00	0.00
	16	3.45	2.56			0		0.00	0.00
P5	12	3.25	2.25			0		0.00	0.00
	17	3.25	2.25			0		0.00	0.00
	16	3.45	2.63			0		0.00	0.00
	18	3.45	2.63			0		0.00	0.00
P6,P7	1	3.25	5.785	8.88		8.88	3.5	15.81	6.23
	2	3.25	5.785	8.88		8.88	3.5	15.81	6.23
	3	5.65				0	3.5	0.00	0.00
	4	5.65				0	3.5	0.00	0.00

Table 6-19- Distribution of the loads- Part 12

Final			
Beam	**Total Dead (KN/m)**	**Live (KN/m)**	**Primeter wall (KN/m)**
1	15.81	6.23	9.00
2	28.98	8.96	-
3	0.00	0.00	9.00
4	13.39	2.76	-
5	27.44	9.88	-
6	28.90	5.98	-
7	23.46	4.65	-
8	13.39	2.76	9.00
9	30.01	12.26	-
10	8.55	1.77	9.00
11	16.60	3.23	-
12	36.66	17.78	-
13	8.05	1.46	9.00

Table 6-20- Distribution of the loads- Part 13

Balcony				
Beam	**Dead (KN/m)**	**Live (KN/m)**	**Dead Moment(KN.m)**	**Live Moment(KN.m)**
5	11.71	6.63	14.99	8.49
9	16.84	9.53	52.63	29.78
12	28.93	16.38	90.41	51.19

Table 6-21- Distribution of the loads- Part 14

F) Sixth Floor:

6th Floor(z=+17.80m)

Primary											
Area Name	Beam	Beam Length (m)	Related Area (m²)	Dead (KN/m²)	Partitions (KN/m²)	Total Dead (KN/m²)	Live (KN/m²)	Snow (KN/m²)	Total Dead (KN/m)	Live (KN/m)	Snow (KN/m)
P1	4	5.65	7.81	10.37	0.21	10.58	2	0.75	14.62	2.76	1.04
	5	6.5	10.55	10.37	0.21	10.58	2	0.75	17.17	3.25	1.22
	6	3.25	5.275	10.37	0.21	10.58	2	0.75	17.17	3.25	1.22
	7	3.25	5.275	10.37	0.21	10.58	2	0.75	17.17	3.25	1.22
	8	5.65	7.81	10.37	0.21	10.58	2	0.75	14.62	2.76	1.04
P2	2	3.25	4.44	10.33	0.21	10.54	2	0.75	14.40	2.73	1.02
	6	3.25	4.44	10.33	0.21	10.54	2	0.75	14.40	2.73	1.02
	9	6.5	8.88	10.33	0.21	10.54	2	0.75	14.40	2.73	1.02
	10	5.75	5.1	10.33	0.21	10.54	2	0.75	9.35	1.77	0.67
	11	5.75	5.1	10.33	0.21	10.54	2	0.75	9.35	1.77	0.67
P3	7	3.25	2.28	10.37	0.21	10.58	2	0.75	7.42	1.40	0.53
	12	3.25	2.28	10.37	0.21	10.58	2	0.75	7.42	1.40	0.53
	11	5.75	4.2	10.37	0.21	10.58	2	0.75	7.73	1.46	0.55
	13	5.75	4.2	10.37	0.21	10.58	2	0.75	7.73	1.46	0.55
P4	9	6.5	6.96			0			0.00	0.00	0.00
	14	6.5	6.96			0			0.00	0.00	0.00
	15	3.45	2.56			0			0.00	0.00	0.00
	16	3.45	2.56			0			0.00	0.00	0.00
P5	12	3.25	2.25			0			0.00	0.00	0.00
	17	3.25	2.25			0			0.00	0.00	0.00
	16	3.45	2.63			0			0.00	0.00	0.00
	18	3.45	2.63			0			0.00	0.00	0.00
P6,P7	1	3.25	5.785	8.88		8.88	3.5		15.81	6.23	0.00
	2	3.25	5.785	8.88		8.88	3.5		15.81	6.23	0.00
	3	5.65				0	3.5		0.00	0.00	0.00
	4	5.65				0	3.5		0.00	0.00	0.00

Table 6-22- Distribution of the loads- Part 15

Final				
Beam	Total Dead (KN/m)	Live (KN/m)	Snow (KN/m)	Primeter wall (KN/m)
1	15.81	6.23	0.00	7.92
2	30.21	8.96	1.02	7.92
3	0.00	0.00	0.00	7.92
4	14.62	2.76	1.04	7.92
5	25.91	3.25	1.22	3.60
6	31.57	5.98	2.24	-
7	24.59	4.65	1.74	-
8	14.62	2.76	1.04	3.60
9	23.05	8.24	1.02	3.60
10	9.35	1.77	0.67	3.60
11	17.08	3.23	1.21	-
12	16.35	7.09	0.53	3.60
13	7.73	1.46	0.55	3.60

Table 6-23- Distribution of the loads- Part 16

Balcony				
Beam	Dead (KN/m)	Live (KN/m)	Dead Moment(KN.m)	Live Moment(KN.m)
5	8.74	0.00	7.96	0.00
9	8.65	5.51	4.33	2.76
12	8.93	5.69	4.47	2.85

Table 6-24- Distribution of the loads- Part 17

G) Seventh Floor (Lift Room):

7th Floor(z=+20.40m)

Primary											
Area Name	**Beam**	**Beam Length (m)**	**Related Area (m^2)**	**Dead (KN/m^2)**	**Partitions (KN/m^2)**	**Total Dead (KN/m^2)**	**Live (KN/m^2)**	**Snow (KN/m^2)**	**Total Dead (KN/m)**	**Live (KN/m)**	**Snow (KN/m)**
P6,P7	**1**	3.25	1.82	8.83	0	8.83	1	0.75	4.94	0.56	0.42
	2	3.25	1.82	8.83	0	8.83	1	0.75	4.94	0.56	0.42
	3	5.65	5.37	8.83	0	8.83	1	0.75	8.39	0.95	0.71
	4	5.65	5.37	8.83	0	8.83	1	0.75	8.39	0.95	0.71

Table 6-25- Distribution of the loads- Part 18

Final			
Beam	**Total Dead (KN/m)**	**Live (KN/m)**	**Snow (KN/m)**
1	4.94	0.56	0.42
2	4.94	0.56	0.42
3	8.39	0.95	0.71
4	8.39	0.95	0.71

Table 6-26- Distribution of the loads- Part 19

7) Spectral Analysis with SAP 2000:

7-1) Spectral Analysis inputs:

As a first step spectral analysis (modal analysis) is performed. There is predefined Modal load case option in the SAP software and the user should just try to open the related widow in the load cases definition part to adjust the desired settings:

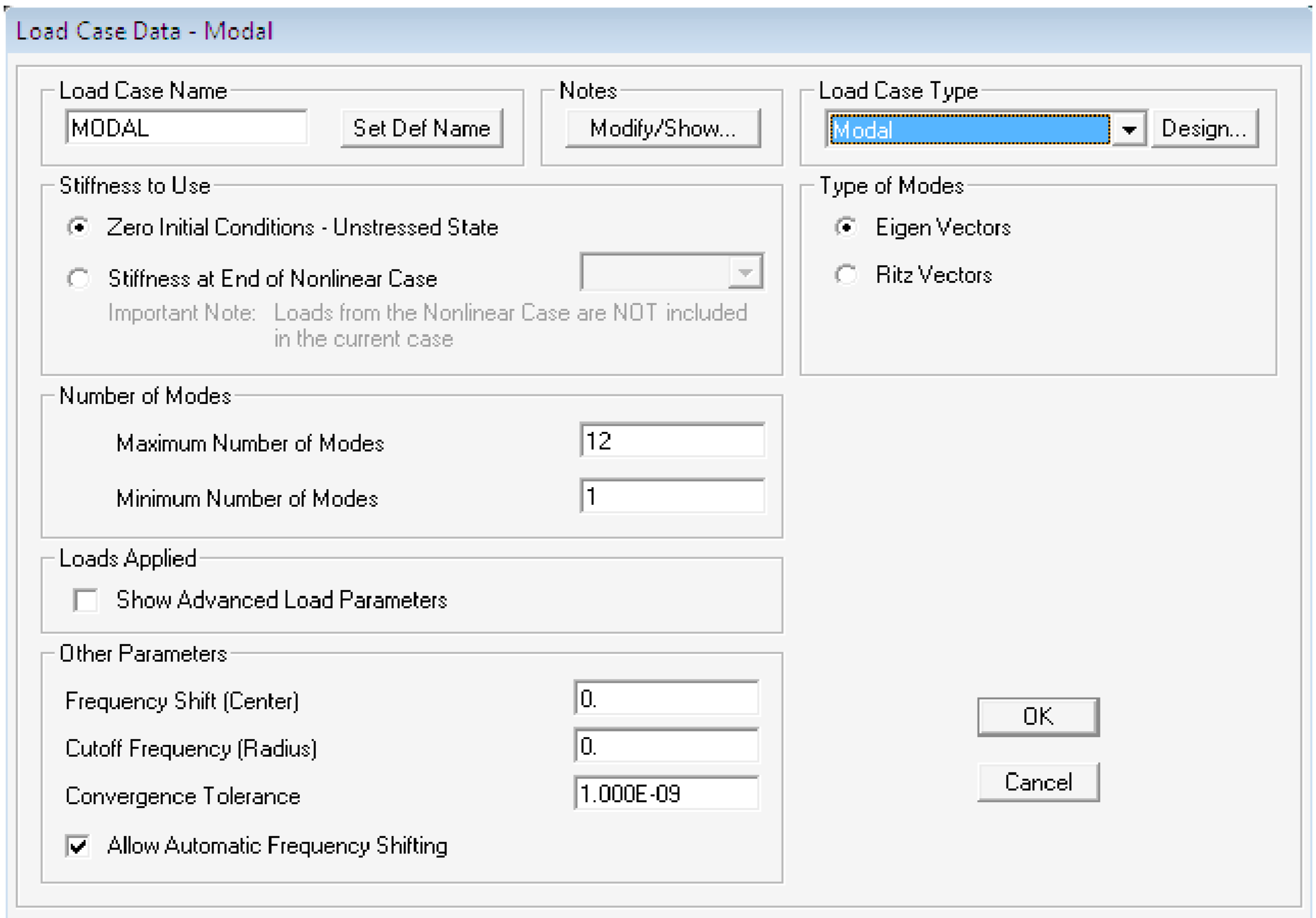

Figure 7-1- Modal Load Cases window in SAP 2000

Now the model is ready for the analysis. Before analysis, the desired load cases for the analysis should be selected to be included in the running process as below:

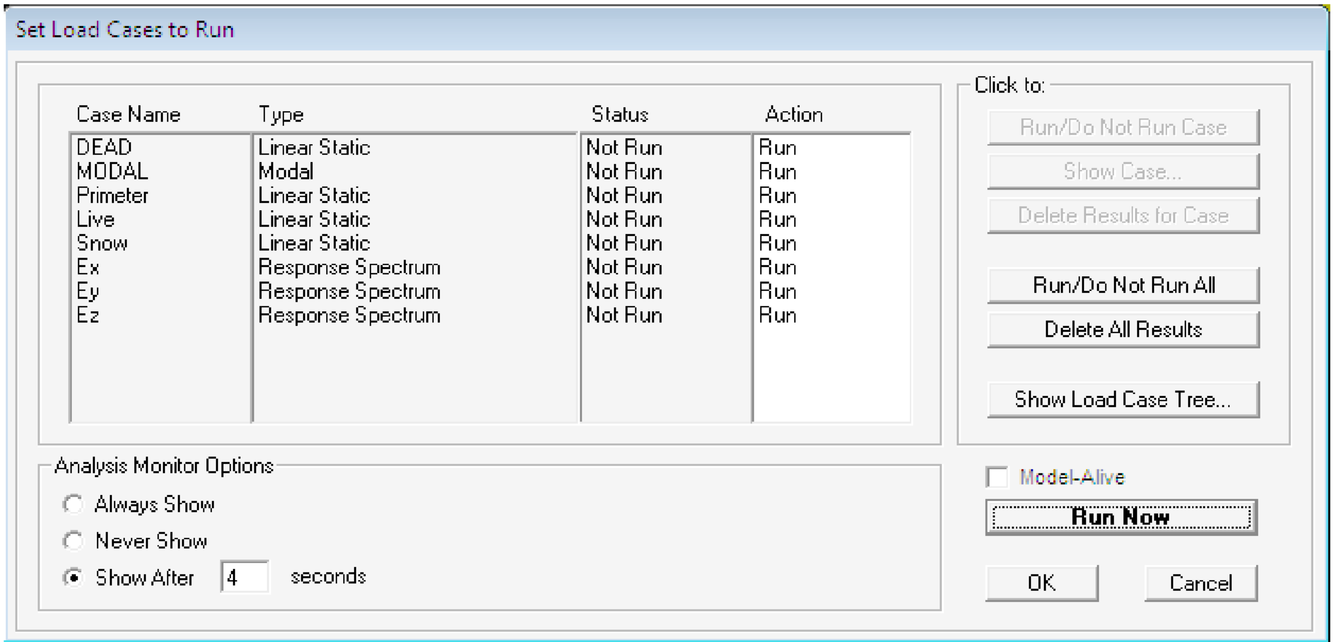

Figure 7-2- Set load Cases to run window in SAP 2000

(9- SAP2000 ultimate software, version 15.0.0)

7-2) Spectral Analysis outputs:

In the following the output of this analysis can be seen. Spectral Analysis Modal shapes and their related period and frequencies are as below:

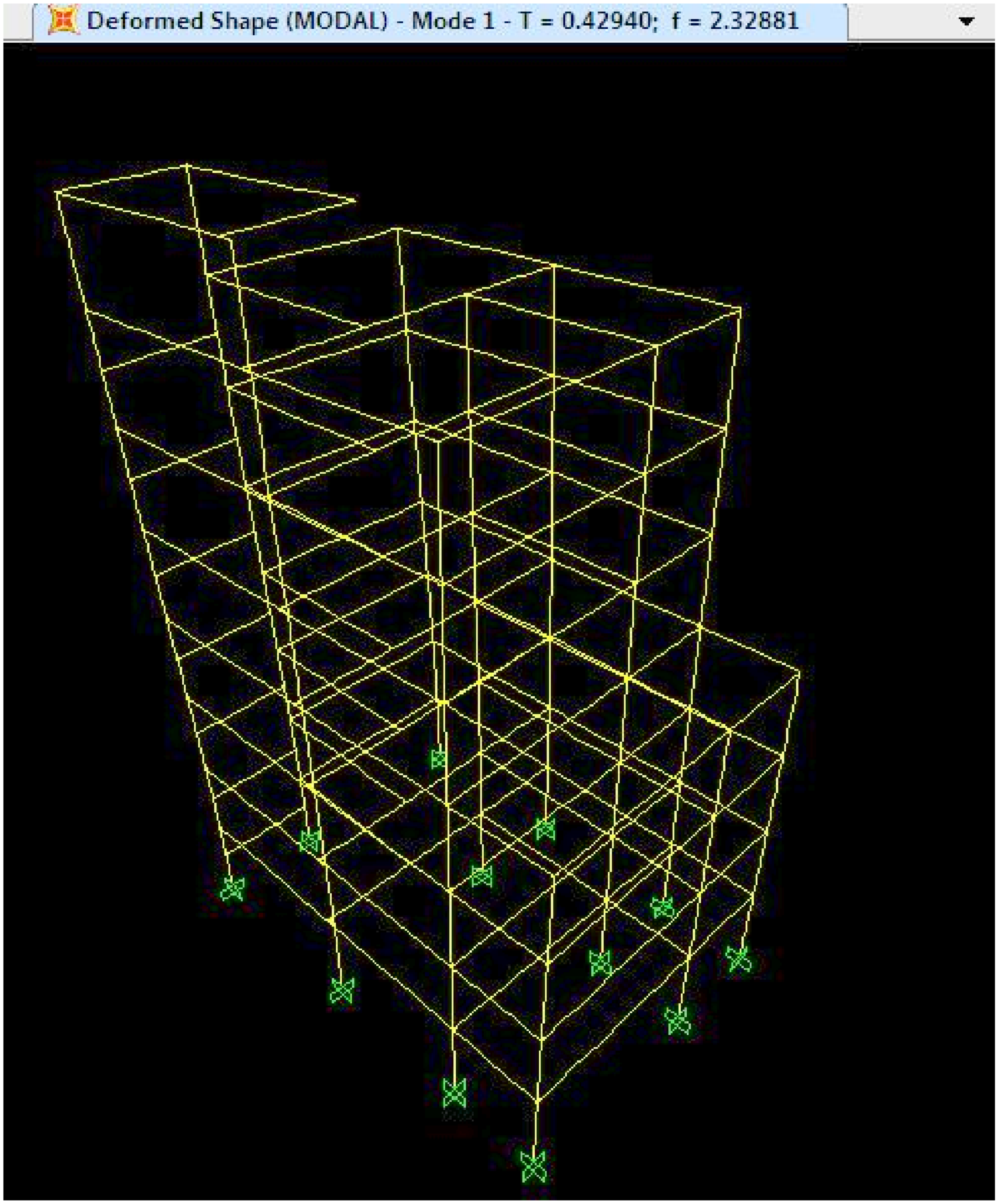

Figure 7-3- Modal Deformed Shape- Mode 1

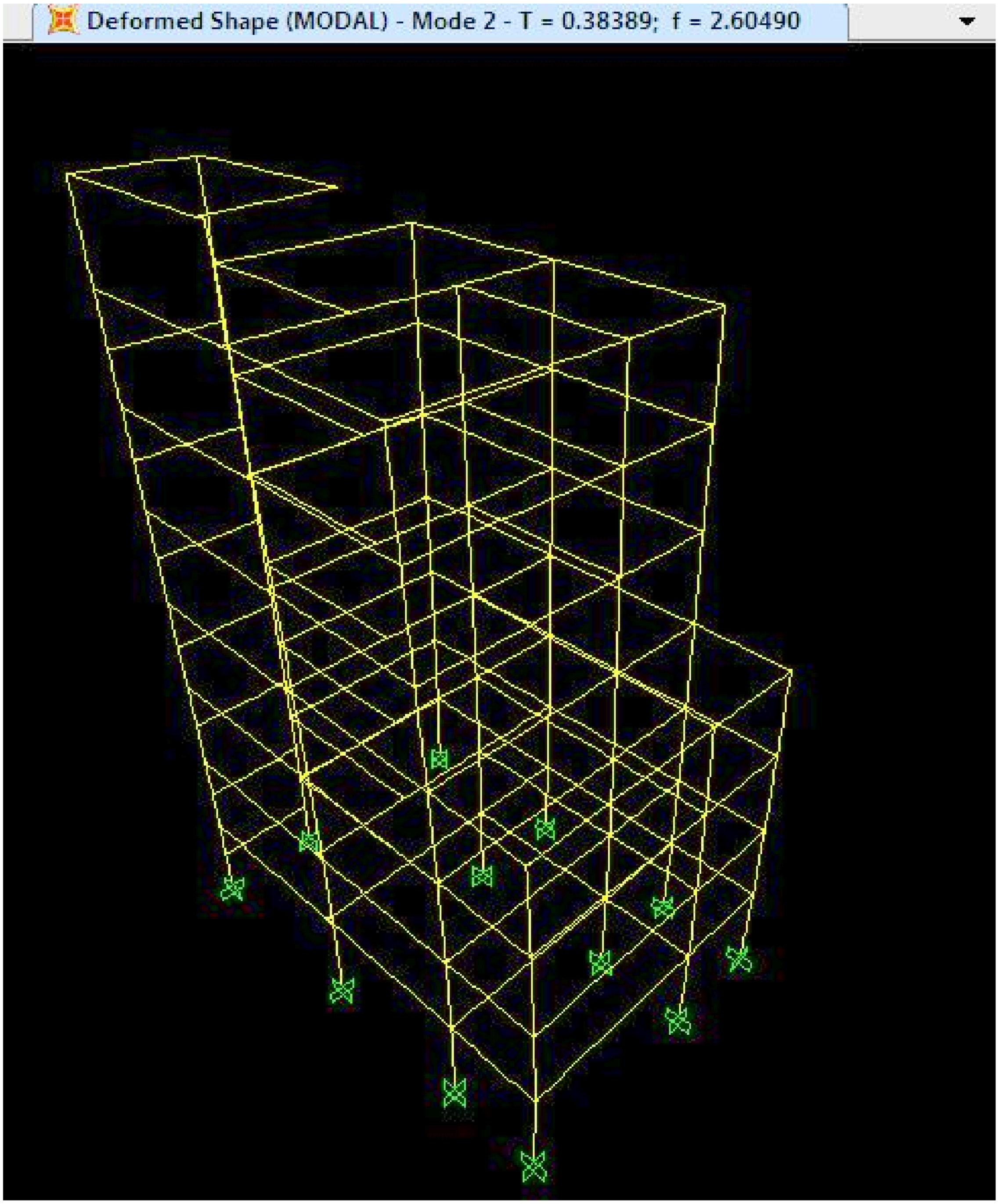

Figure 7-4- Modal Deformed Shape- Mode 2

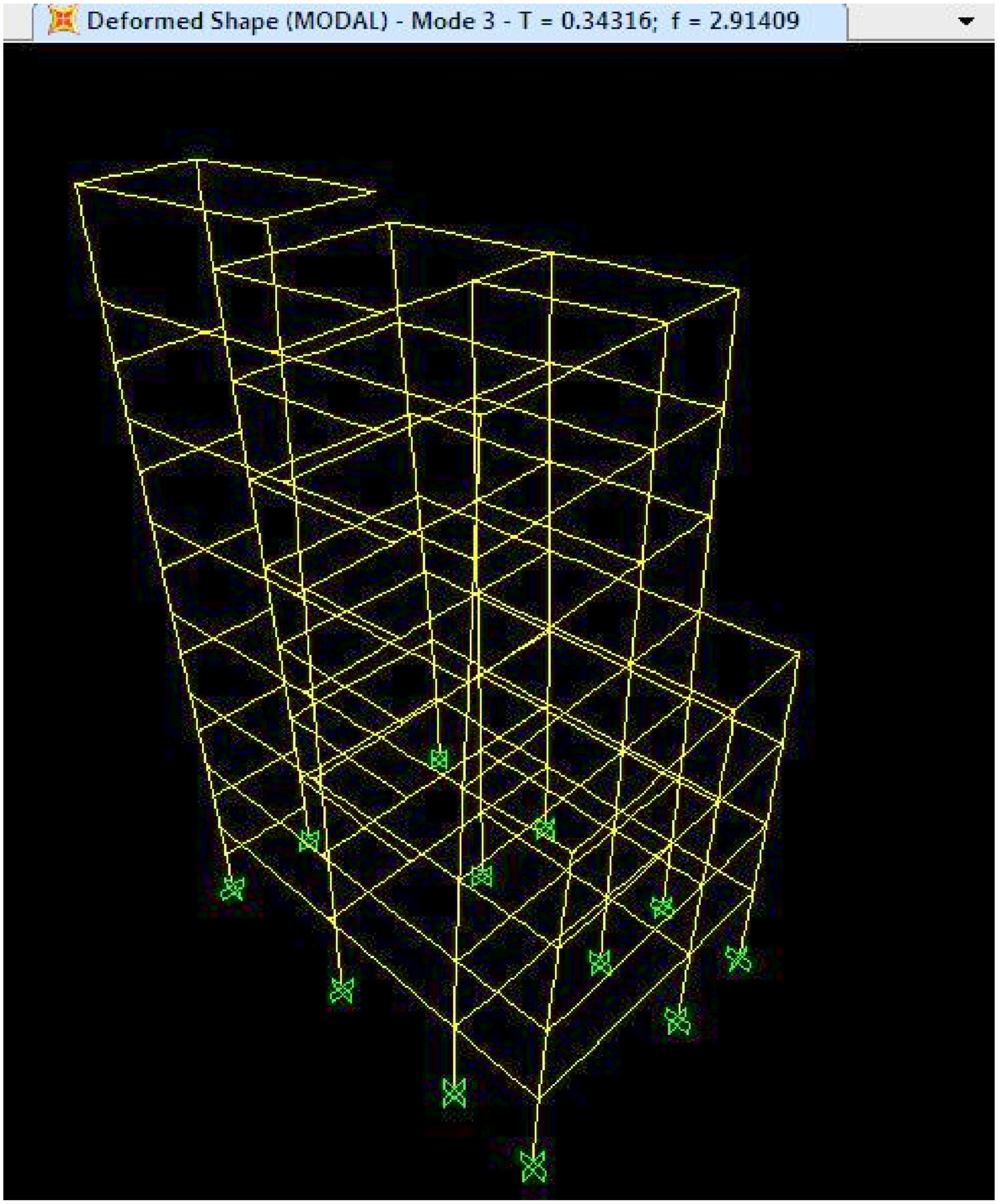

Figure 7-5- Modal Deformed Shape- Mode 3

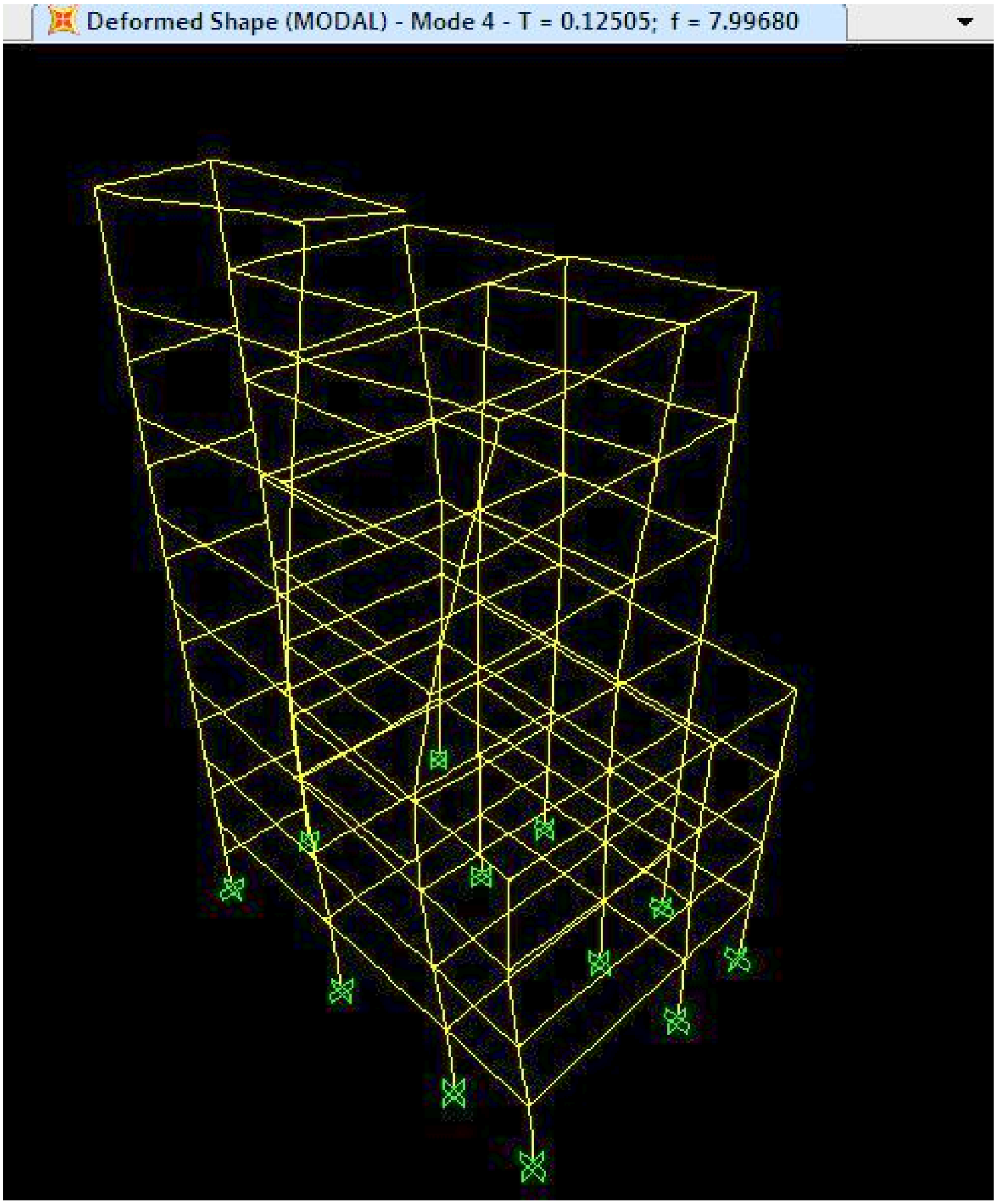

Figure 7-6- Modal Deformed Shape- Mode 4

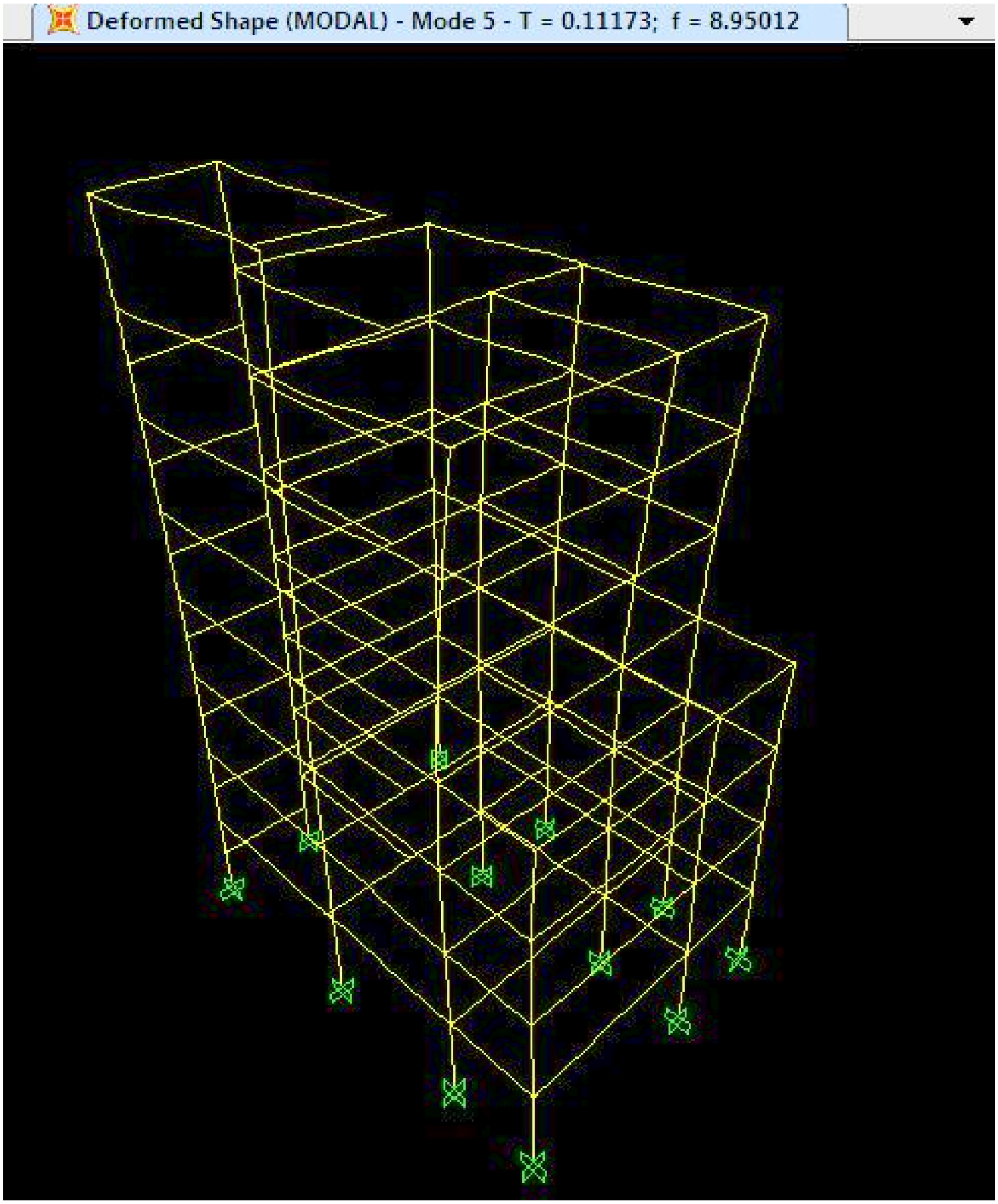

Figure 7-7- Modal Deformed Shape- Mode 5

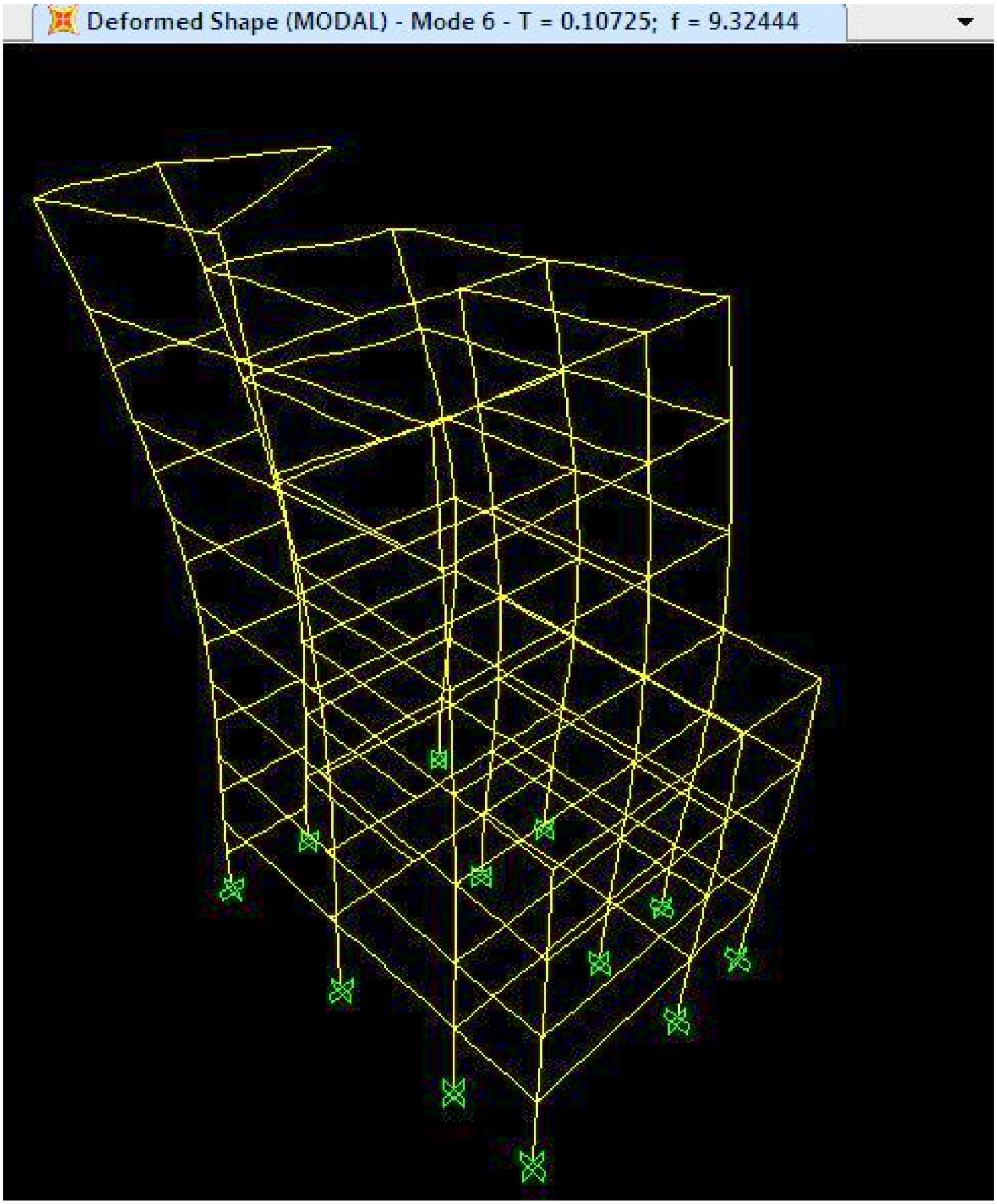

Figure 7-8- Modal Deformed Shape- Mode 6

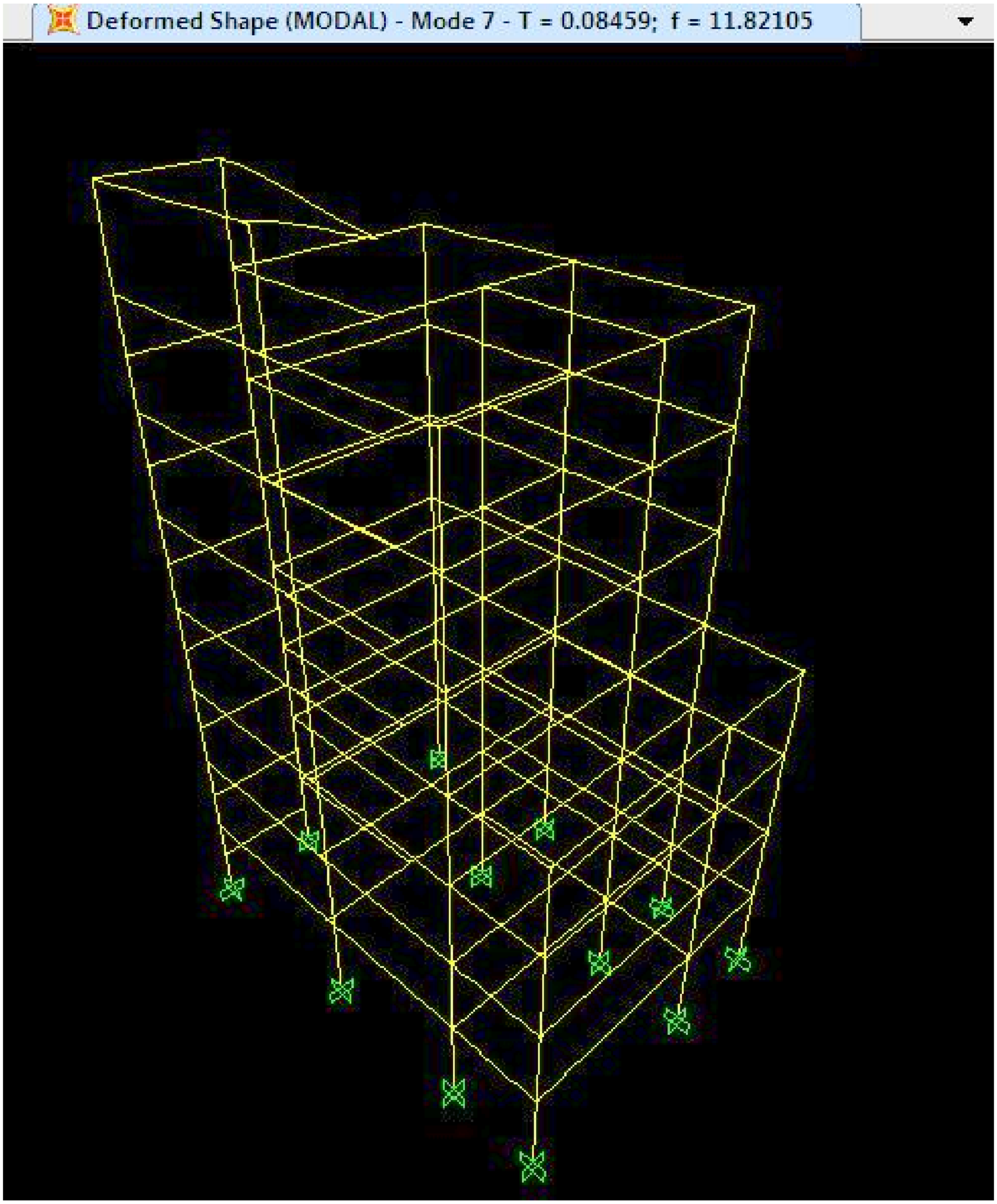

Figure 7-9- Modal Deformed Shape- Mode 7

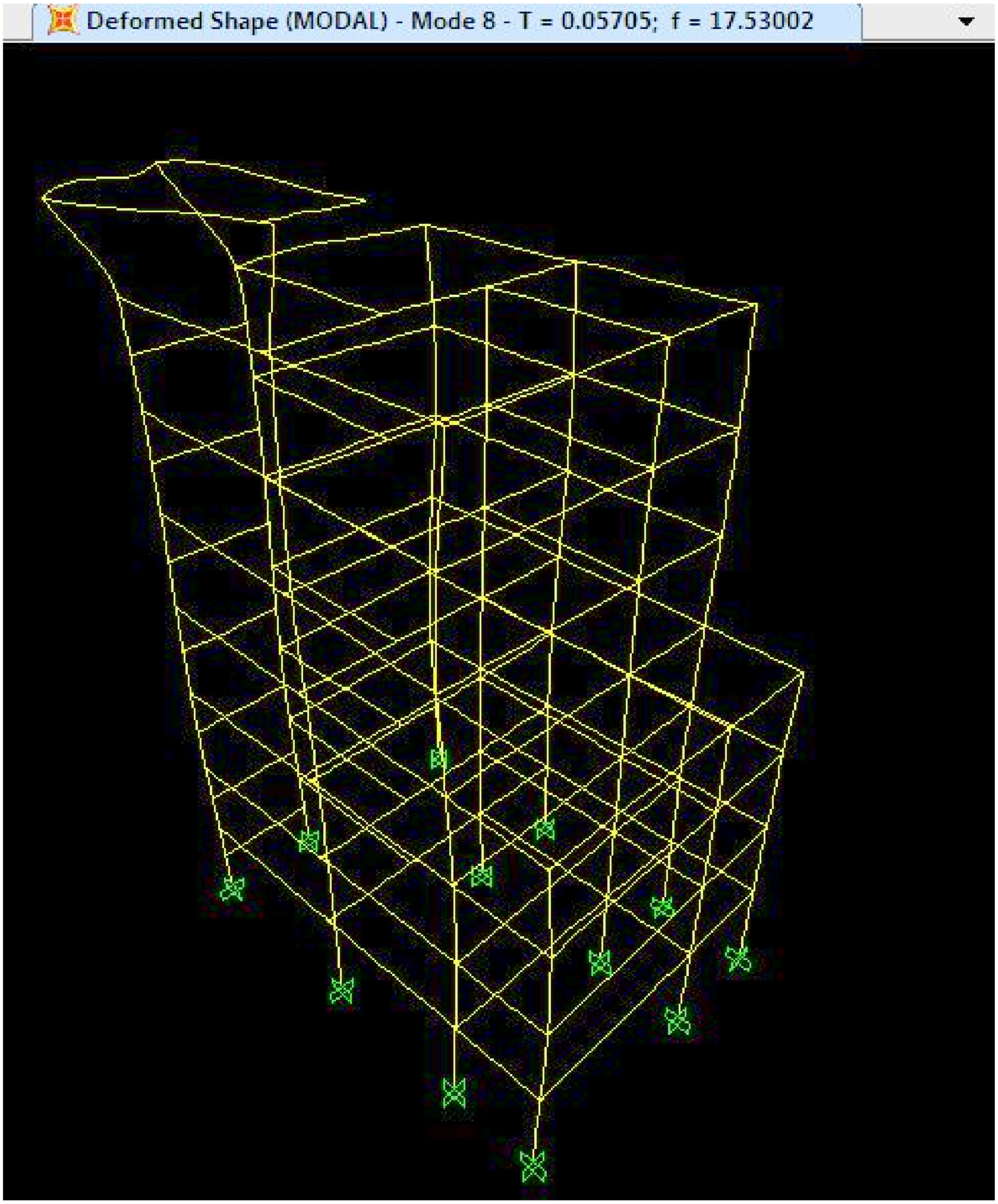

Figure 7-10- Modal Deformed Shape- Mode 8

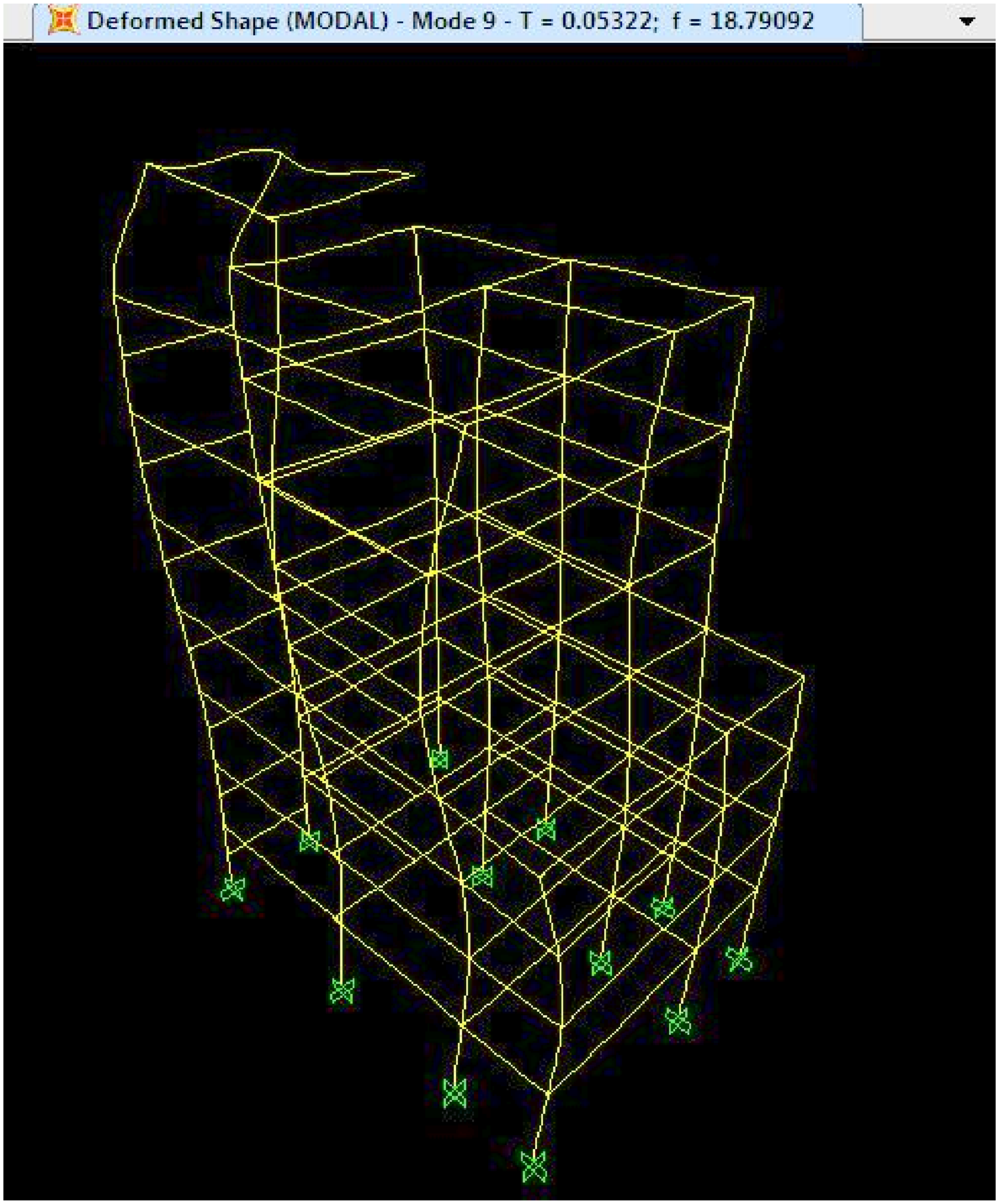

Figure 7-11- Modal Deformed Shape- Mode 9

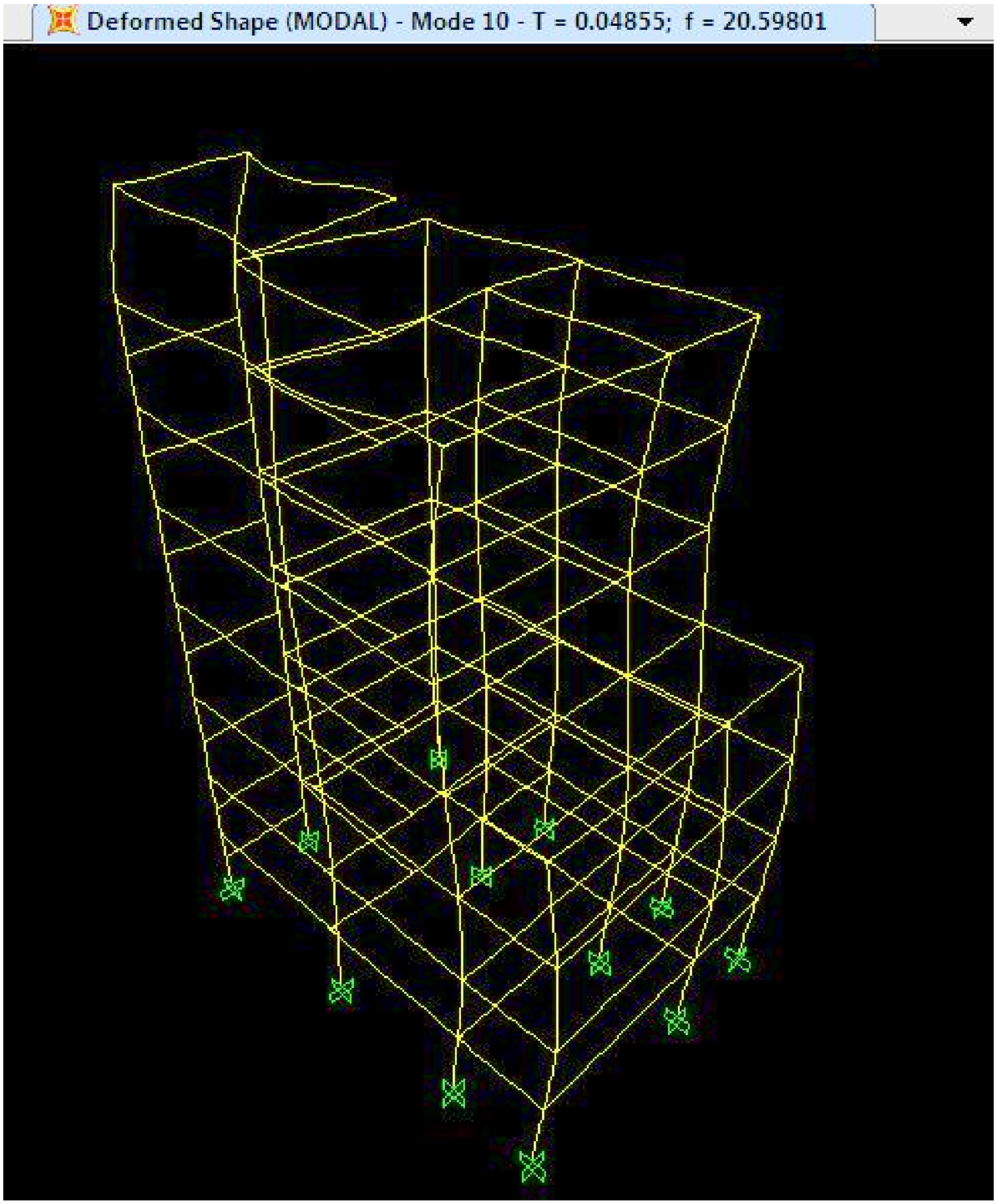

Figure 7-12- Modal Deformed Shape- Mode 10

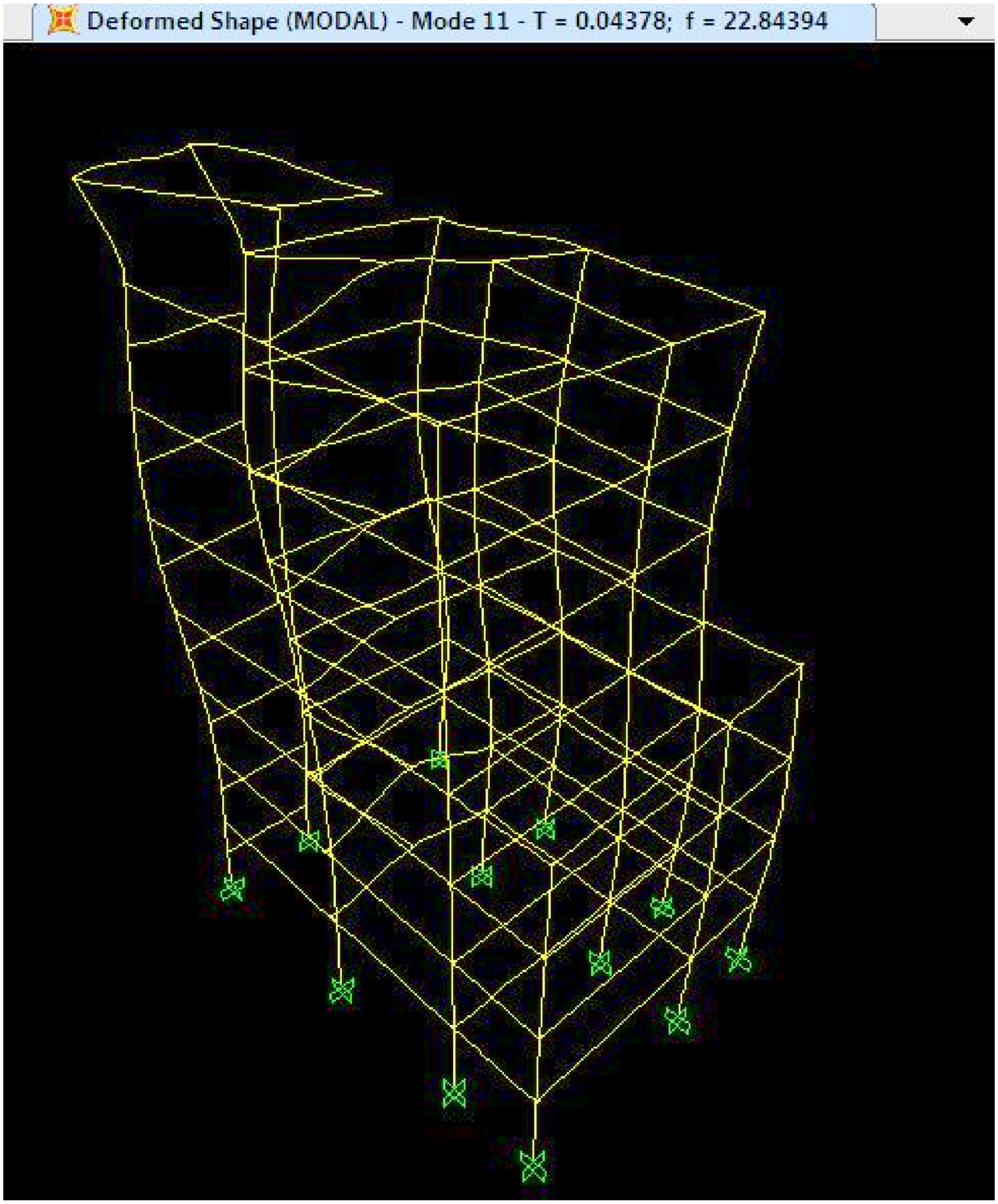

Figure 7-13- Modal Deformed Shape- Mode 11

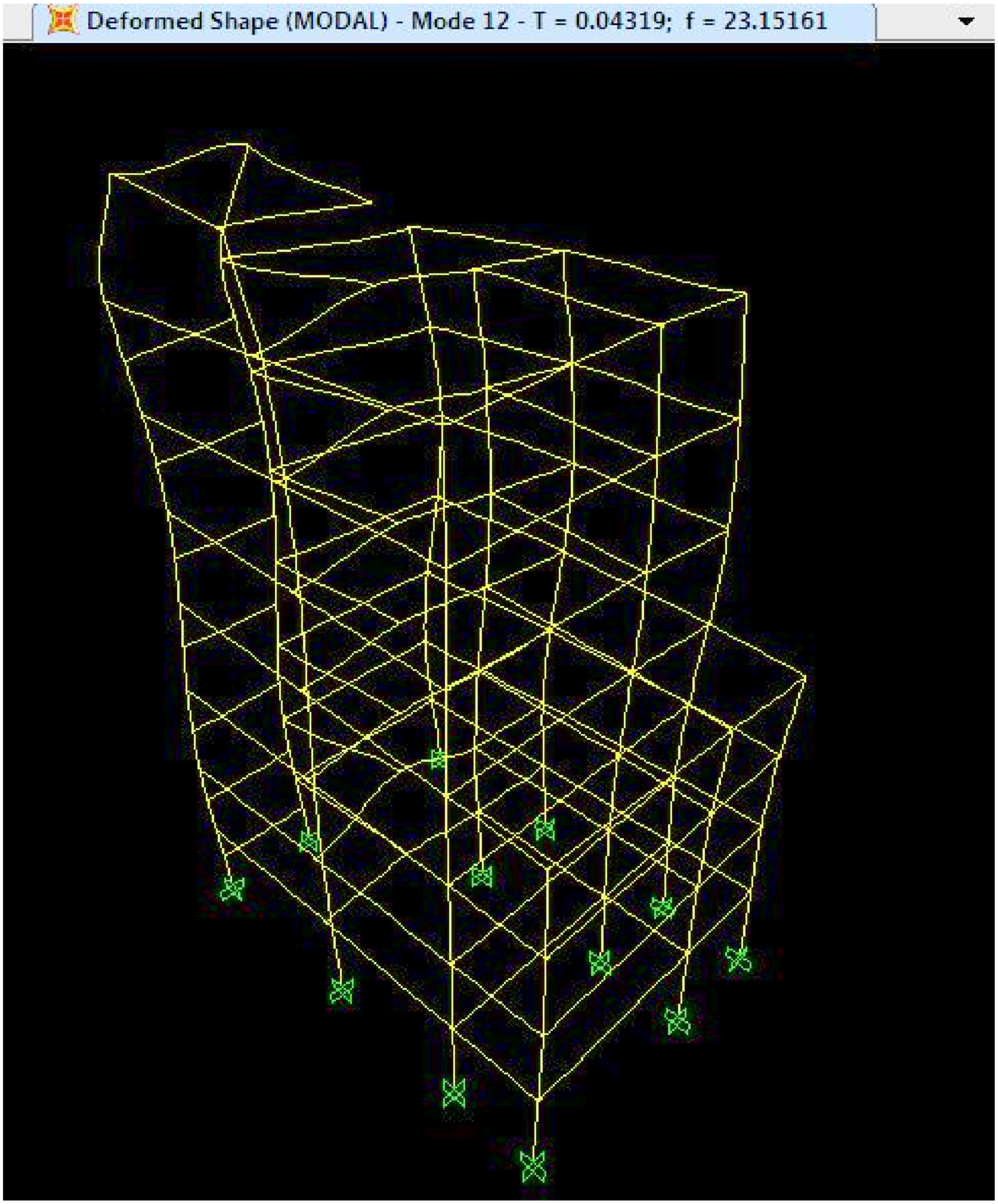

Figure 7-14- Modal Deformed Shape- Mode 12

The numerical analysis information can be summarizes as below:

OutputCase	ItemType	Item	Static	Dynamic
			Percent	Percent
MODAL	Acceleration	UX	99.9547	88.7067
MODAL	Acceleration	UY	99.9525	88.0547
MODAL	Acceleration	UZ	55.6541	29.2646

Table 7-1- Modal Load Participation Ratios

OutputCase	StepType	StepNum	Period	UX	UY	UZ	SumUX	SumUY
			Sec					
MODAL	Mode	1.000000	0.429403	0.48665	0.04383	0.00088	0.48665	0.04383
MODAL	Mode	2.000000	0.383892	0.10467	0.49573	6.745E-06	0.59132	0.53956
MODAL	Mode	3.000000	0.343160	0.05842	0.11464	5.234E-06	0.64974	0.65419
MODAL	Mode	4.000000	0.125050	0.05802	0.00423	0.00041	0.70776	0.65842
MODAL	Mode	5.000000	0.111730	0.03940	0.11645	0.00065	0.74716	0.77487
MODAL	Mode	6.000000	0.107245	0.07046	0.03963	0.00072	0.81762	0.81450
MODAL	Mode	7.000000	0.084595	0.00164	0.00116	0.00443	0.81925	0.81566
MODAL	Mode	8.000000	0.057045	8.266E-08	0.00025	9.770E-06	0.81925	0.81591
MODAL	Mode	9.000000	0.053217	0.04567	0.00128	0.00027	0.86492	0.81719
MODAL	Mode	10.000000	0.048548	0.00787	0.05185	0.00355	0.87279	0.86903
MODAL	Mode	11.000000	0.043775	0.00647	0.00666	0.15782	0.87926	0.87569
MODAL	Mode	12.000000	0.043194	0.00781	0.00486	0.12389	0.88707	0.88055

Table 7-2- Modal Participating Mass Ratios, Part 1 of 3

OutputCase	StepType	StepNum	SumUZ	RX	RY	RZ	SumRX	SumRY
MODAL	Mode	1.000000	0.00088	0.03302	0.59254	0.39632	0.03302	0.59254
MODAL	Mode	2.000000	0.00088	0.43374	0.10973	0.14456	0.46676	0.70227
MODAL	Mode	3.000000	0.00089	0.09582	0.04804	0.12907	0.56259	0.75031
MODAL	Mode	4.000000	0.00130	0.00028	1.547E-05	1.840E-05	0.56287	0.75033
MODAL	Mode	5.000000	0.00195	0.00011	8.291E-06	0.00316	0.56297	0.75034
MODAL	Mode	6.000000	0.00267	0.00078	9.681E-05	0.14961	0.56376	0.75043
MODAL	Mode	7.000000	0.00710	0.00255	0.00011	0.00238	0.56631	0.75055
MODAL	Mode	8.000000	0.00711	2.332E-05	1.491E-05	0.01259	0.56633	0.75056
MODAL	Mode	9.000000	0.00738	4.812E-05	0.00010	0.00712	0.56638	0.75066
MODAL	Mode	10.000000	0.01093	0.00029	8.437E-05	1.811E-07	0.56667	0.75075
MODAL	Mode	11.000000	0.16875	0.07676	0.02272	0.02214	0.64343	0.77347
MODAL	Mode	12.000000	0.29265	0.04324	0.02586	0.02591	0.68667	0.79934

Table 7-3- Modal Participating Mass Ratios, Part 2 of 3

OutputCase	StepType	StepNum	SumRZ
MODAL	Mode	1.000000	0.39632
MODAL	Mode	2.000000	0.54089
MODAL	Mode	3.000000	0.66996
MODAL	Mode	4.000000	0.66997
MODAL	Mode	5.000000	0.67314
MODAL	Mode	6.000000	0.82275
MODAL	Mode	7.000000	0.82513
MODAL	Mode	8.000000	0.83772
MODAL	Mode	9.000000	0.84483
MODAL	Mode	10.000000	0.84483
MODAL	Mode	11.000000	0.86698
MODAL	Mode	12.000000	0.89289

Table 7-4- Modal Participating Mass Ratios, Part 3 of 3

OutputCase	StepType	StepNum	Period	UX	UY	UZ	RX	RY
			Sec	KN-s2	KN-s2	KN-s2	KN-m-s2	KN-m-s2
MODAL	Mode	1.000000	0.429403	15.815765	4.746434	0.670672	-55.547074	203.151397
MODAL	Mode	2.000000	0.383892	7.334781	-15.962574	-0.058882	201.321747	87.420730
MODAL	Mode	3.000000	0.343160	-5.479569	-7.676079	0.051867	94.626300	-57.847292
MODAL	Mode	4.000000	0.125050	5.461015	-1.474477	0.458265	5.114327	-1.038049
MODAL	Mode	5.000000	0.111730	-4.500106	-7.736585	0.578592	3.141199	0.759908
MODAL	Mode	6.000000	0.107245	-6.017933	4.513081	0.609258	8.543420	-2.596679
MODAL	Mode	7.000000	0.084595	-0.917173	0.772612	1.509292	15.442826	-2.812343
MODAL	Mode	8.000000	0.057045	0.006518	-0.354978	0.070865	1.476067	1.018895
MODAL	Mode	9.000000	0.053217	-4.844997	0.810871	0.371444	2.120506	-2.688008
MODAL	Mode	10.000000	0.048548	2.011444	5.162203	1.351017	5.230811	-2.424185
MODAL	Mode	11.000000	0.043775	-1.822962	1.849619	-9.006747	-84.693071	39.781619
MODAL	Mode	12.000000	0.043194	2.003409	-1.580434	-7.979956	-63.561313	42.443863

Table 7-5- Modal Participation Factors, Part 1 of 2

OutputCase	StepType	StepNum	RZ	ModalMass	ModalStiff
			KN-m-s2	KN-m-s2	KN-m
MODAL	Mode	1.000000	-153.529695	1.0000	214.10606
MODAL	Mode	2.000000	-92.725203	1.0000	267.88070
MODAL	Mode	3.000000	-87.615435	1.0000	335.24772
MODAL	Mode	4.000000	-1.046009	1.0000	2524.60088
MODAL	Mode	5.000000	13.714749	1.0000	3162.40467
MODAL	Mode	6.000000	94.331237	1.0000	3432.45755
MODAL	Mode	7.000000	11.899493	1.0000	5516.60361
MODAL	Mode	8.000000	-27.358873	1.0000	12131.77701
MODAL	Mode	9.000000	20.571514	1.0000	13939.78426
MODAL	Mode	10.000000	0.103788	1.0000	16749.82930
MODAL	Mode	11.000000	36.289893	1.0000	20601.62942
MODAL	Mode	12.000000	-39.258147	1.0000	21160.30763

Table 7-6- Modal Participation Factors, Part 2 of 2

OutputCase	StepType	StepNum	Period	Frequency	CircFreq	Eigenvalue
			Sec	Cyc/sec	rad/sec	rad2/sec2
MODAL	Mode	1.000000	0.429403	2.3288E+00	1.4632E+01	2.1411E+02
MODAL	Mode	2.000000	0.383892	2.6049E+00	1.6367E+01	2.6788E+02
MODAL	Mode	3.000000	0.343160	2.9141E+00	1.8310E+01	3.3525E+02
MODAL	Mode	4.000000	0.125050	7.9968E+00	5.0245E+01	2.5246E+03
MODAL	Mode	5.000000	0.111730	8.9501E+00	5.6235E+01	3.1624E+03
MODAL	Mode	6.000000	0.107245	9.3244E+00	5.8587E+01	3.4325E+03
MODAL	Mode	7.000000	0.084595	1.1821E+01	7.4274E+01	5.5166E+03
MODAL	Mode	8.000000	0.057045	1.7530E+01	1.1014E+02	1.2132E+04
MODAL	Mode	9.000000	0.053217	1.8791E+01	1.1807E+02	1.3940E+04
MODAL	Mode	10.000000	0.048548	2.0598E+01	1.2942E+02	1.6750E+04
MODAL	Mode	11.000000	0.043775	2.2844E+01	1.4353E+02	2.0602E+04
MODAL	Mode	12.000000	0.043194	2.3152E+01	1.4547E+02	2.1160E+04

Table 7-7- Modal Periods and Frequencies

OutputCase	ModalCase	StepType	StepNum	Period	DampRatio	U1Acc	U2Acc	U3Acc
				Sec		m/sec2	m/sec2	m/sec2
Ex	MODAL	Mode	1.000000	0.429403	0.0500	0.13714	0.00000	0.00000
Ex	MODAL	Mode	2.000000	0.383892	0.0500	0.13714	0.00000	0.00000
Ex	MODAL	Mode	3.000000	0.343160	0.0500	0.13714	0.00000	0.00000
Ex	MODAL	Mode	4.000000	0.125050	0.0500	0.13562	0.00000	0.00000
Ex	MODAL	Mode	5.000000	0.111730	0.0500	0.13481	0.00000	0.00000
Ex	MODAL	Mode	6.000000	0.107245	0.0500	0.13454	0.00000	0.00000
Ex	MODAL	Mode	7.000000	0.084595	0.0500	0.13316	0.00000	0.00000
Ex	MODAL	Mode	8.000000	0.057045	0.0500	0.13148	0.00000	0.00000
Ex	MODAL	Mode	9.000000	0.053217	0.0500	0.13124	0.00000	0.00000
Ex	MODAL	Mode	10.000000	0.048548	0.0500	0.13096	0.00000	0.00000
Ex	MODAL	Mode	11.000000	0.043775	0.0500	0.13067	0.00000	0.00000
Ex	MODAL	Mode	12.000000	0.043194	0.0500	0.13063	0.00000	0.00000
Ey	MODAL	Mode	1.000000	0.429403	0.0500	0.00000	0.13714	0.00000
Ey	MODAL	Mode	2.000000	0.383892	0.0500	0.00000	0.13714	0.00000
Ey	MODAL	Mode	3.000000	0.343160	0.0500	0.00000	0.13714	0.00000
Ey	MODAL	Mode	4.000000	0.125050	0.0500	0.00000	0.13562	0.00000
Ey	MODAL	Mode	5.000000	0.111730	0.0500	0.00000	0.13481	0.00000
Ey	MODAL	Mode	6.000000	0.107245	0.0500	0.00000	0.13454	0.00000
Ey	MODAL	Mode	7.000000	0.084595	0.0500	0.00000	0.13316	0.00000
Ey	MODAL	Mode	8.000000	0.057045	0.0500	0.00000	0.13148	0.00000
Ey	MODAL	Mode	9.000000	0.053217	0.0500	0.00000	0.13124	0.00000
Ey	MODAL	Mode	10.000000	0.048548	0.0500	0.00000	0.13096	0.00000
Ey	MODAL	Mode	11.000000	0.043775	0.0500	0.00000	0.13067	0.00000
Ey	MODAL	Mode	12.000000	0.043194	0.0500	0.00000	0.13063	0.00000
Ez	MODAL	Mode	1.000000	0.429403	0.0500	0.00000	0.00000	0.02057
Ez	MODAL	Mode	2.000000	0.383892	0.0500	0.00000	0.00000	0.02057
Ez	MODAL	Mode	3.000000	0.343160	0.0500	0.00000	0.00000	0.02057
Ez	MODAL	Mode	4.000000	0.125050	0.0500	0.00000	0.00000	0.02034
Ez	MODAL	Mode	5.000000	0.111730	0.0500	0.00000	0.00000	0.02022
Ez	MODAL	Mode	6.000000	0.107245	0.0500	0.00000	0.00000	0.02018
Ez	MODAL	Mode	7.000000	0.084595	0.0500	0.00000	0.00000	0.01997
Ez	MODAL	Mode	8.000000	0.057045	0.0500	0.00000	0.00000	0.01972
Ez	MODAL	Mode	9.000000	0.053217	0.0500	0.00000	0.00000	0.01969
Ez	MODAL	Mode	10.000000	0.048548	0.0500	0.00000	0.00000	0.01964
Ez	MODAL	Mode	11.000000	0.043775	0.0500	0.00000	0.00000	0.01960
Ez	MODAL	Mode	12.000000	0.043194	0.0500	0.00000	0.00000	0.01959

Table 7-8- Response Spectrum Modal Information, Part 1 of 2

OutputCase	StepType	StepNum	U1Amp	U2Amp	U3Amp
			m	m	m
Ex	Mode	1.000000	0.010131	0.000000	0.000000
Ex	Mode	2.000000	0.003755	0.000000	0.000000
Ex	Mode	3.000000	-0.002242	0.000000	0.000000
Ex	Mode	4.000000	0.000293	0.000000	0.000000
Ex	Mode	5.000000	-0.000192	0.000000	0.000000
Ex	Mode	6.000000	-0.000236	0.000000	0.000000
Ex	Mode	7.000000	-0.000022	0.000000	0.000000
Ex	Mode	8.000000	7.064E-08	0.000000	0.000000
Ex	Mode	9.000000	-0.000046	0.000000	0.000000
Ex	Mode	10.000000	0.000016	0.000000	0.000000
Ex	Mode	11.000000	-0.000012	0.000000	0.000000
Ex	Mode	12.000000	0.000012	0.000000	0.000000
Ey	Mode	1.000000	0.000000	0.003040	0.000000
Ey	Mode	2.000000	0.000000	-0.008172	0.000000
Ey	Mode	3.000000	0.000000	-0.003140	0.000000
Ey	Mode	4.000000	0.000000	-0.000079	0.000000
Ey	Mode	5.000000	0.000000	-0.000330	0.000000
Ey	Mode	6.000000	0.000000	0.000177	0.000000
Ey	Mode	7.000000	0.000000	0.000019	0.000000
Ey	Mode	8.000000	0.000000	-3.847E-06	0.000000
Ey	Mode	9.000000	0.000000	7.634E-06	0.000000
Ey	Mode	10.000000	0.000000	0.000040	0.000000
Ey	Mode	11.000000	0.000000	0.000012	0.000000
Ey	Mode	12.000000	0.000000	-9.757E-06	0.000000
Ez	Mode	1.000000	0.000000	0.000000	0.000064
Ez	Mode	2.000000	0.000000	0.000000	-4.522E-06
Ez	Mode	3.000000	0.000000	0.000000	3.183E-06
Ez	Mode	4.000000	0.000000	0.000000	3.693E-06
Ez	Mode	5.000000	0.000000	0.000000	3.700E-06
Ez	Mode	6.000000	0.000000	0.000000	3.582E-06
Ez	Mode	7.000000	0.000000	0.000000	5.465E-06
Ez	Mode	8.000000	0.000000	0.000000	1.152E-07
Ez	Mode	9.000000	0.000000	0.000000	5.246E-07
Ez	Mode	10.000000	0.000000	0.000000	1.584E-06
Ez	Mode	11.000000	0.000000	0.000000	-8.569E-06
Ez	Mode	12.000000	0.000000	0.000000	-7.390E-06

Table 7-9- Response Spectrum Modal Information, Part 2 of 2

(9- SAP2000 ultimate software, version 15.0.0)

8) Pushover Analysis with SAP 2000:

8-1) Pushover Analysis inputs:

For pushover analysis there are some actions that should be done before analysis such as hinge definition for beams and columns and assigning them to the related frames, definition of the pushover load cases…

8-1-1) Hinge definition:

In the define hinge properties window, two different hinge properties should be defined, one for the beams and the other one for the columns:

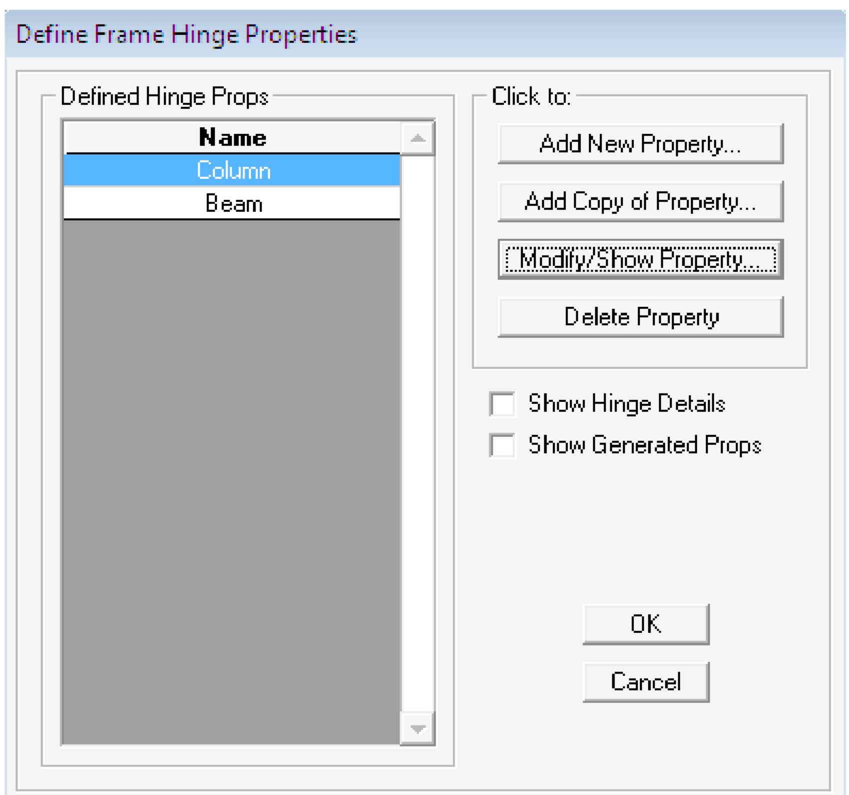

Figure 8-1- Define Frame Hinge Properties Window in SAP 2000

For the definition of the beams' hinges, The M3 moment should be selected:

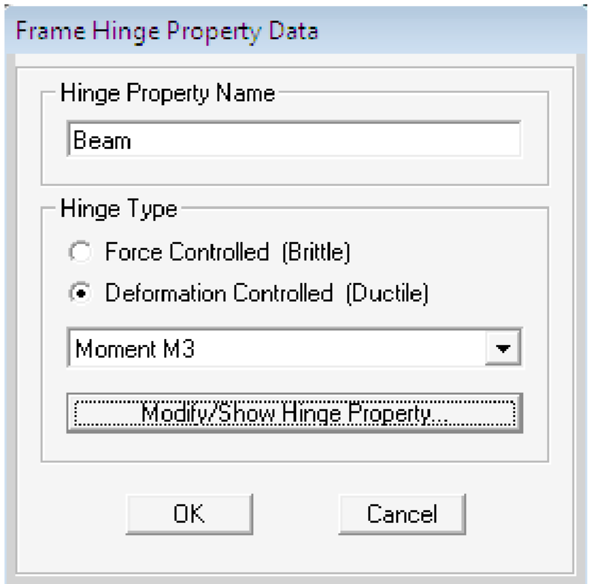

Figure 8-2- Beam Hinge Property Data Window in SAP 2000

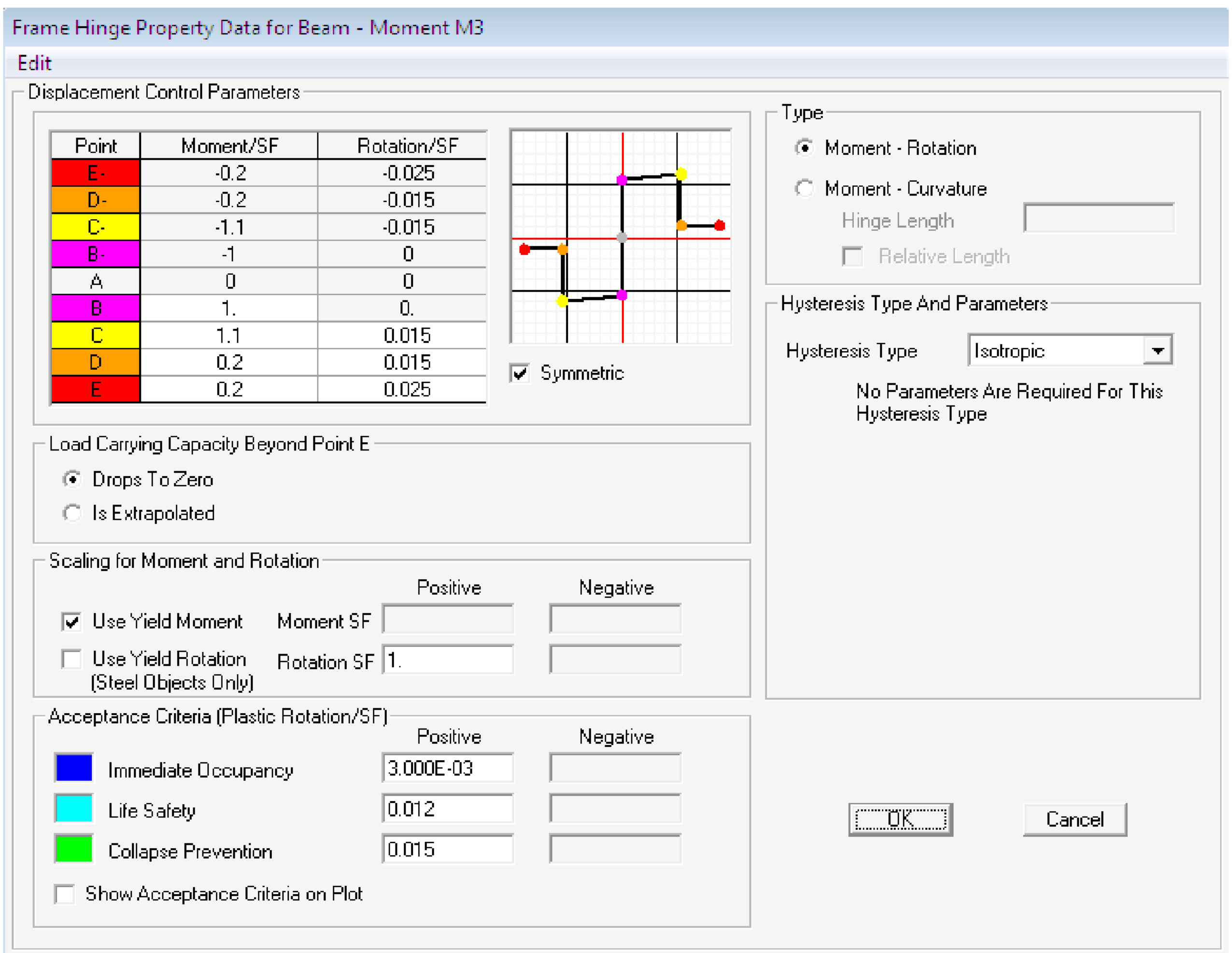

Figure 8-3- Frame Hinge Property Data for Beam Window in SAP 2000

For the definition of the columns' hinges, The P-M2-M3 should be selected:

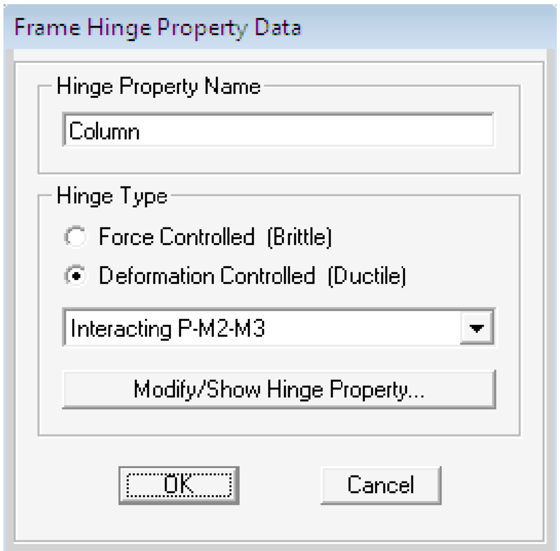

Figure 8-4- Column Hinge Property Data Window in SAP 2000

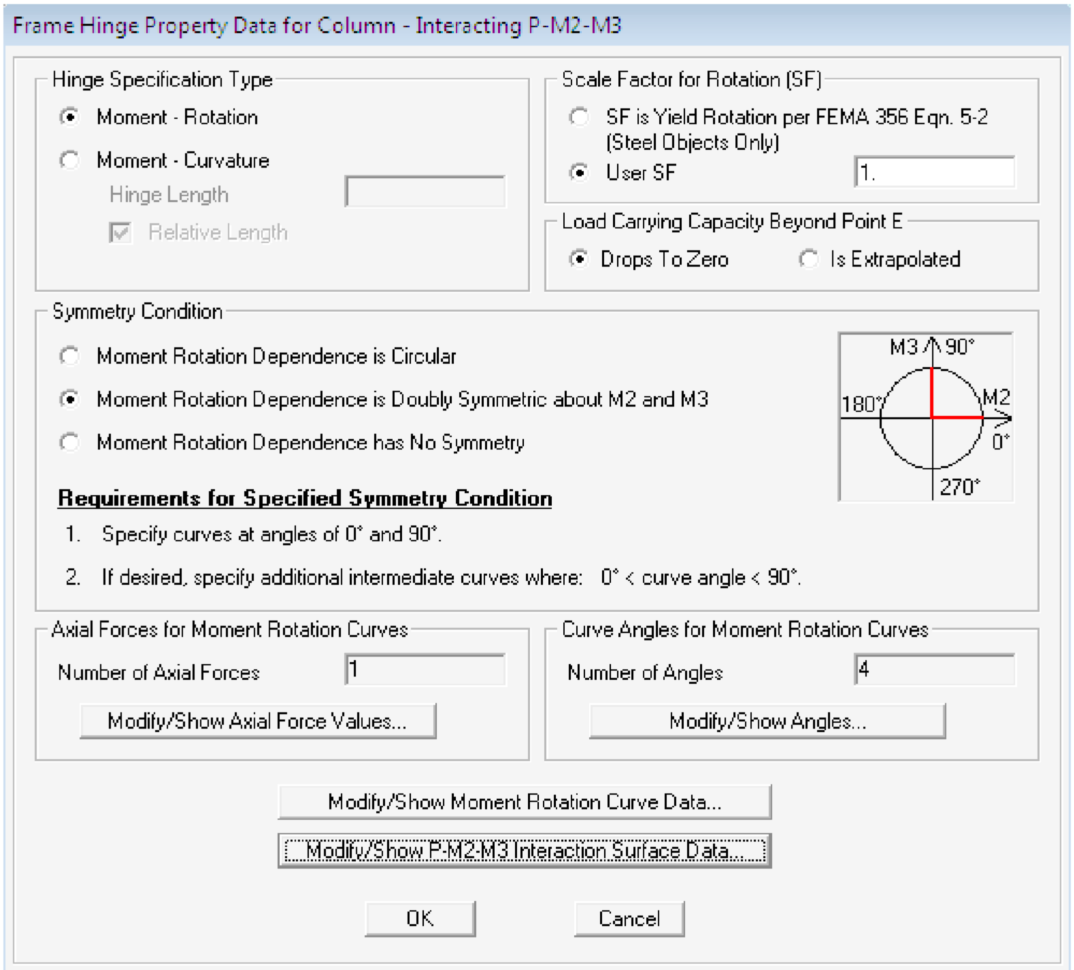

Figure 8-5- Frame Hinge Property Data for Column Window in SAP 2000

Now is the time to assign the defined hinges to the desired frames. At this step, all of the beams should be selected and then the “frame hinge assignments” window should be opened. In this window “beam” hinge property should be selected and then two relative distances of 0.01 and 0.99 should be entered for the location of the hinges at the both endings:

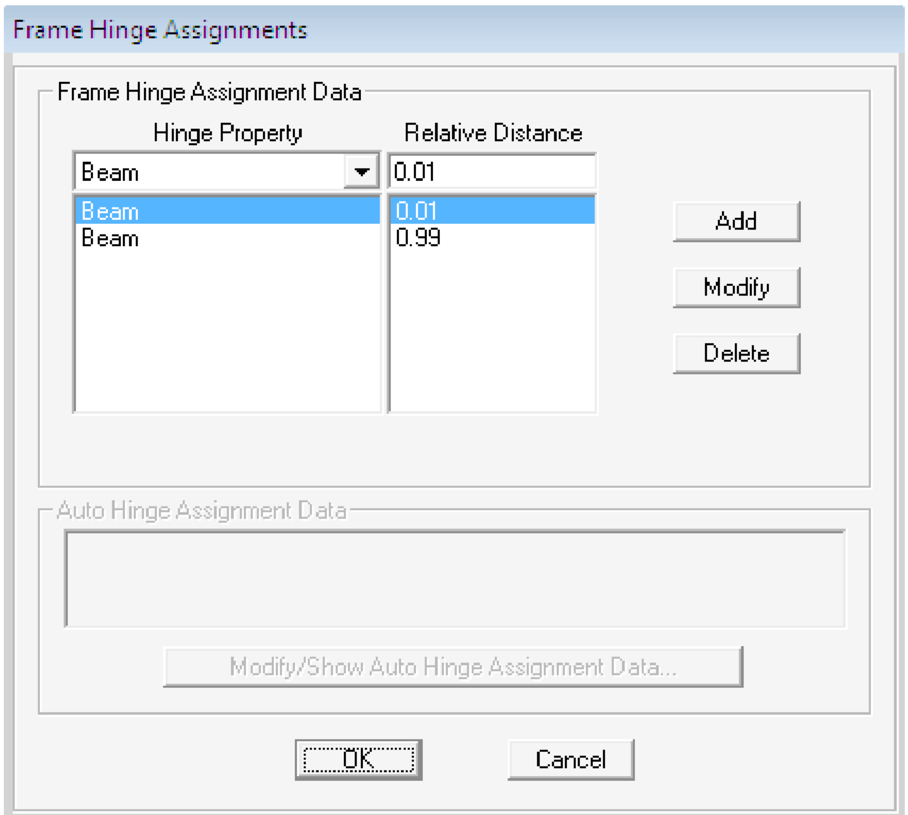

Figure 8-6- Beam Hinge Assignments Window in SAP 2000

Now, all of the columns should be selected and then the “frame hinge assignments” window should be opened. In this window “column” hinge property should be selected and then two relative distances of 0.01 and 0.99 should be entered for the location of the critical sections where hinges are expected to form at both ends:

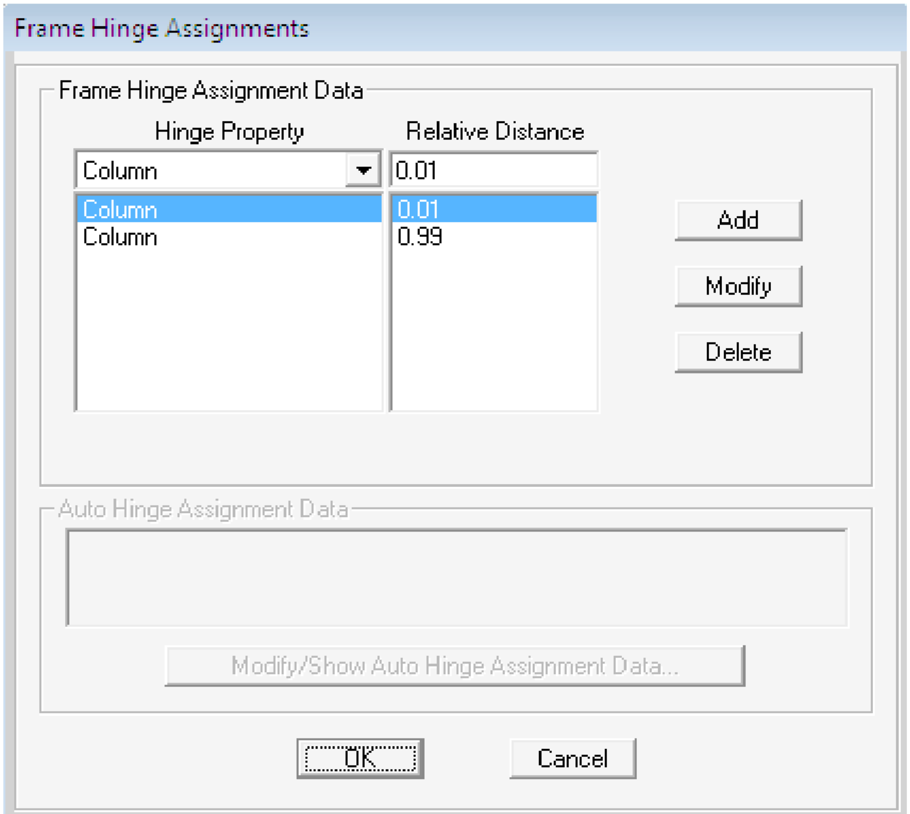

Figure 8-7- Column Hinge Assignments Window in SAP 2000

At the last step for hinge property definition, all of the frames should be selected and then from the “frame hinge assignment overwrites” window the “auto subdivide line objects at hinges” checkbox should be checked:

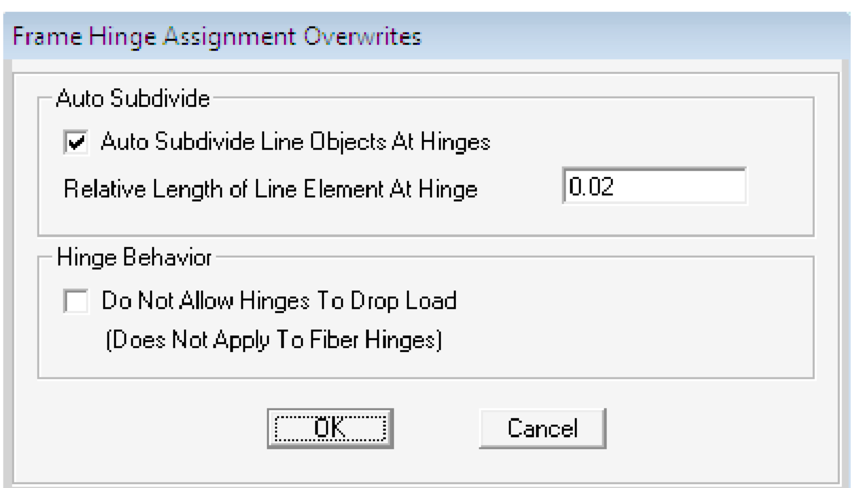

Figure 8-8- Frame Hinge Assignments Overwrites Window in SAP 2000

8-1-2) Pushover load case definition:

As it was mentioned in the previous chapters, in this kind of analysis, the gravity loads should be applied on the structure and the lateral loads should be applied on the structure step by step till it reaches to a specific target displacement. For the gravity loads, all of the dead loads that are composed of dead loads and perimeter heavy walls self weight and a percentage of the live loads should be combined with each other and applied to the structure for the required primary gravity loads. For this issue a new load combination should be defined as below:

D + P + 0.3 L

This new load combination and also the pushover load cases can be defined by help of the "define load cases" window. Before definition of these new load cases, it should be reminded that this kind of analysis is a "Non linear" analysis so the type of the "Dead, in fill walls at the perimeter, Live" load cases that are defined before should be changed to the nonlinear type:

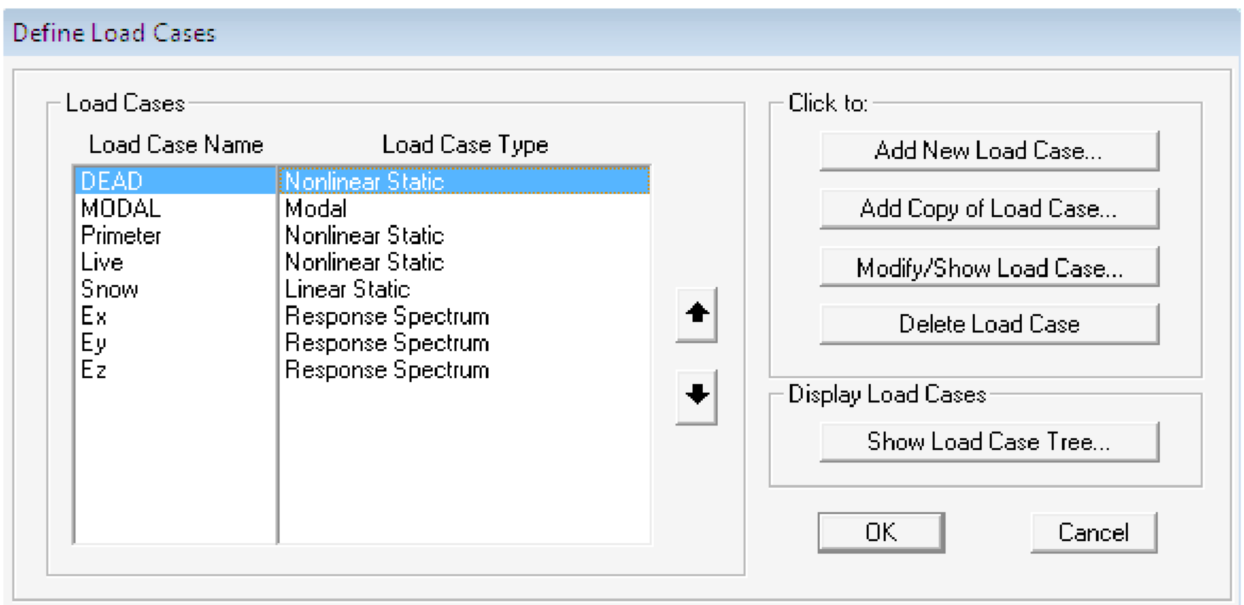

Figure 8-9- Define Load Cases Window in SAP 2000

Now the above mentioned new load case as a new nonlinear load case can be defined too. In this step the pushover load cases can be defined, one for the "X" direction (push x) and the other one for the "Y" direction (push y).

For the “push x” load case, the static nonlinear option should be selected and in the “initial conditions” part, “continue from state at end of nonlinear case” checkbox should be checked and from the dropdown list “D + P + 0.3L” load case should be selected. The “Ux” as a “Accel” load case should be selected. At the “load application” window the “Displacement control” option should be checked, the “monitored displacement” with a value of 0.3m should be selected for one of the joints that is located at the last storey with a DOF in the “U1” direction (joint No. 81). Finally in the “Results saved” window, the “Multiple states” should be checked:

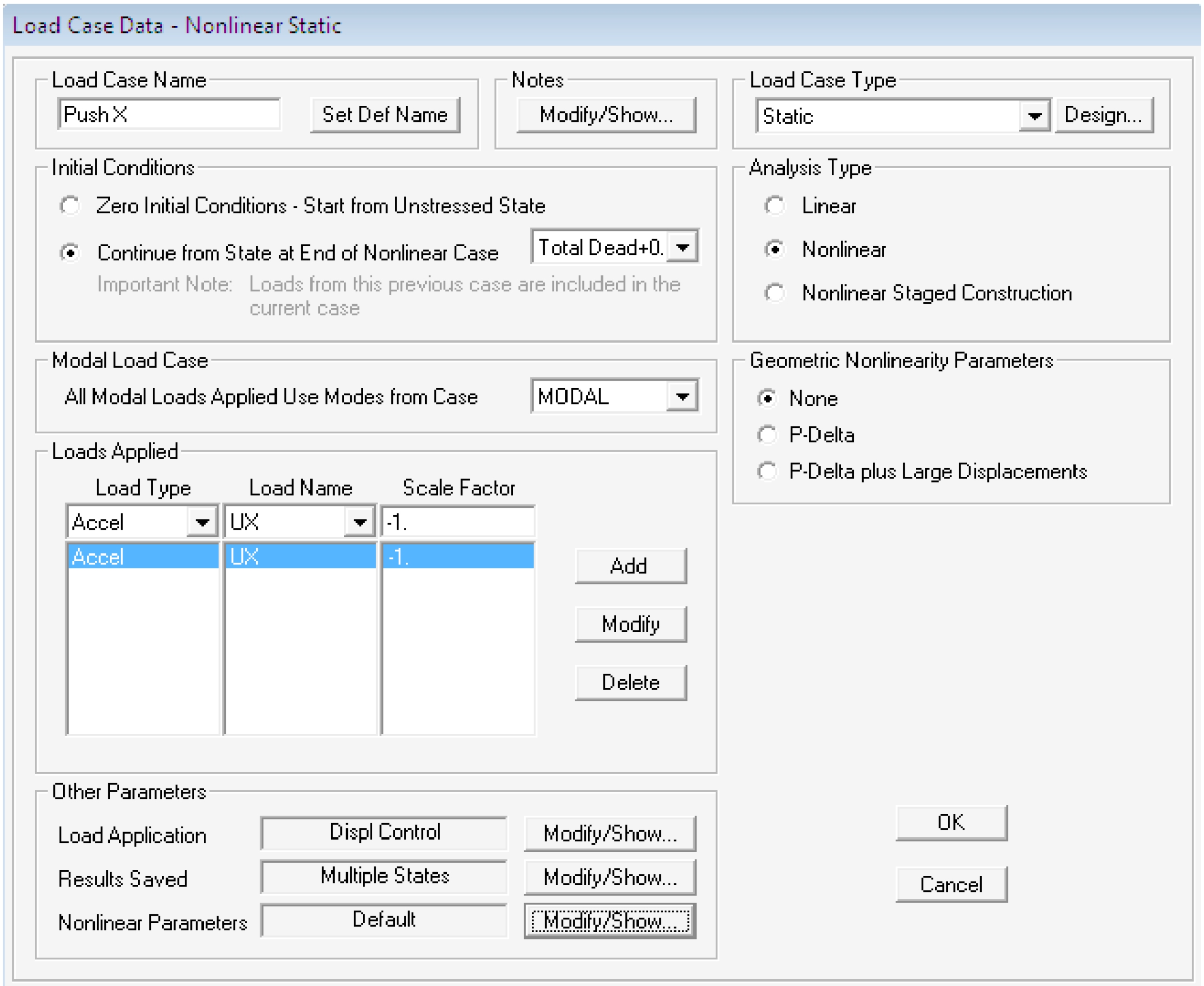

Figure 8-10- Pushover load case definition in X direction Window in SAP 2000

For definition of the “push y” load case, the same steps should be done. The only differences are these:

The “Uy” as a “Accel” load case should be selected. The “monitored displacement” with a value of 0.3m should be selected for one of the joints that is located at the last storey with a DOF in the “U2” direction (joint No. 81):

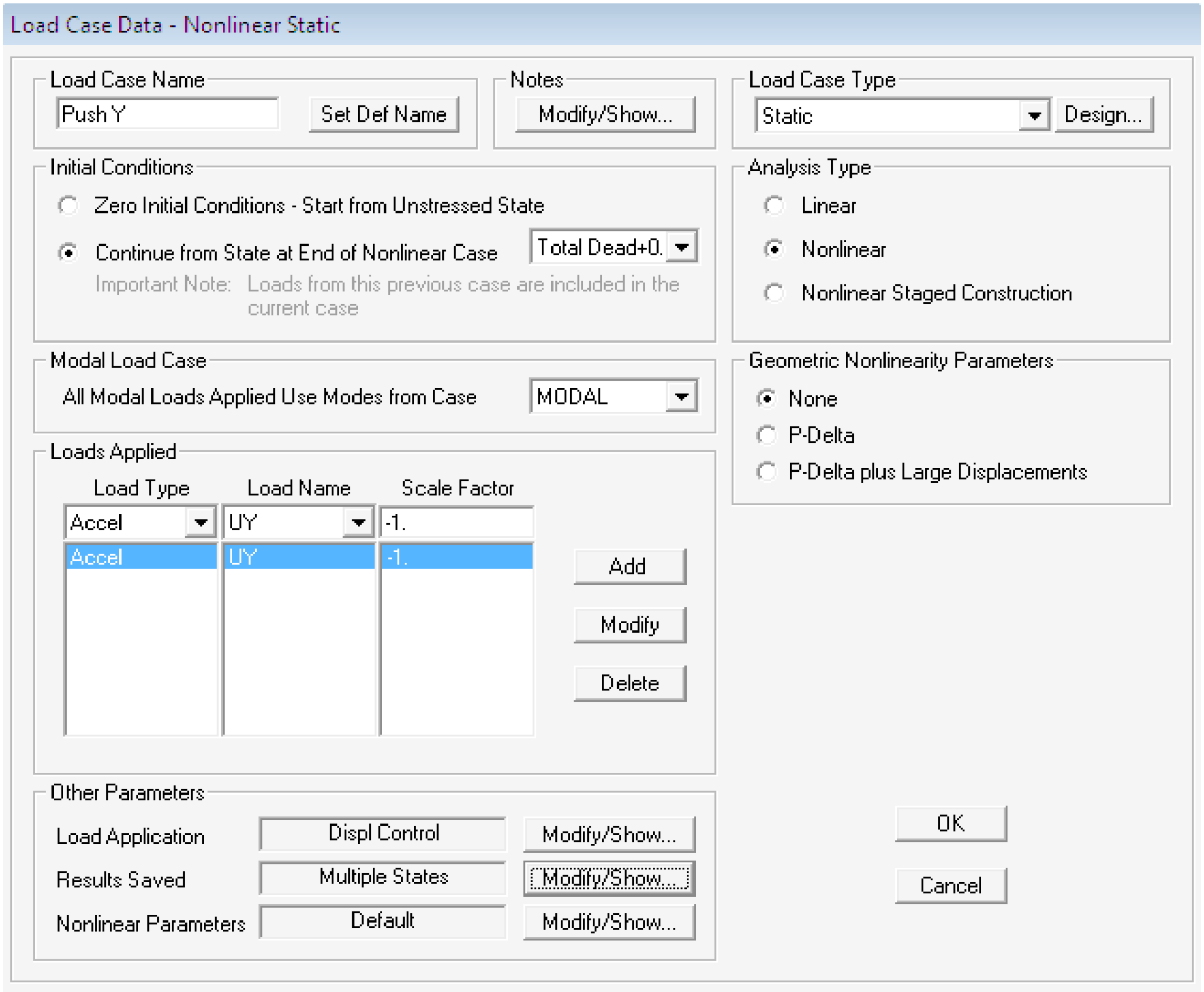

Figure 8-11- Pushover load case definition in Y direction Window in SAP 2000

Now the model is ready for analysis. In the “set load cases to run” window, only the “Dead, Perimeter, Live, D + P + 0.3 L, push x and push y” load cases should be selected for analysis:

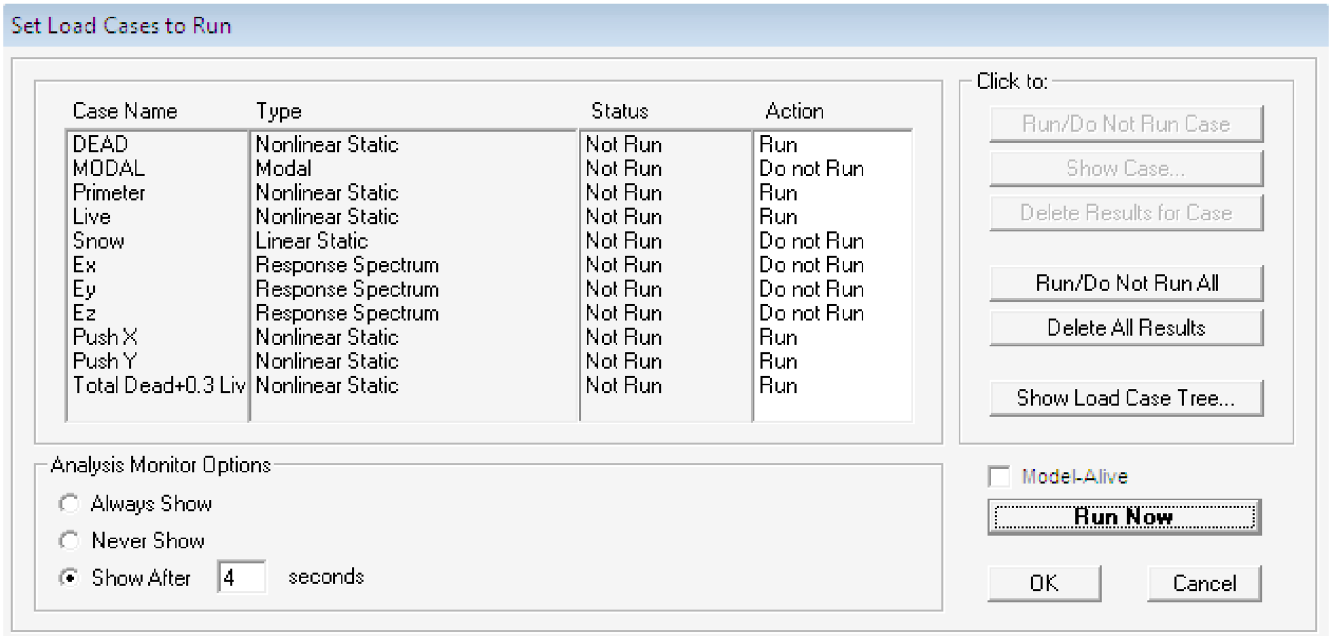

Figure 8-12- Set Load Cases to Run Window in SAP 2000

(9- SAP2000 ultimate software, version 15.0.0)

8-2) Pushover Analysis outputs:

The analysis outputs are then presented. First of all the process of initiation of the hinges in the structure is shown in different steps graphically. By using these photos, the gradual process of creation of the hinges in whole of the structure can be seen. Next the push over graphs such as base reaction-displacement and ATC-40 capacity spectrum and their related tabular outputs will be presented.

8-2-1) X direction:

8-2-1-1) Hinges:

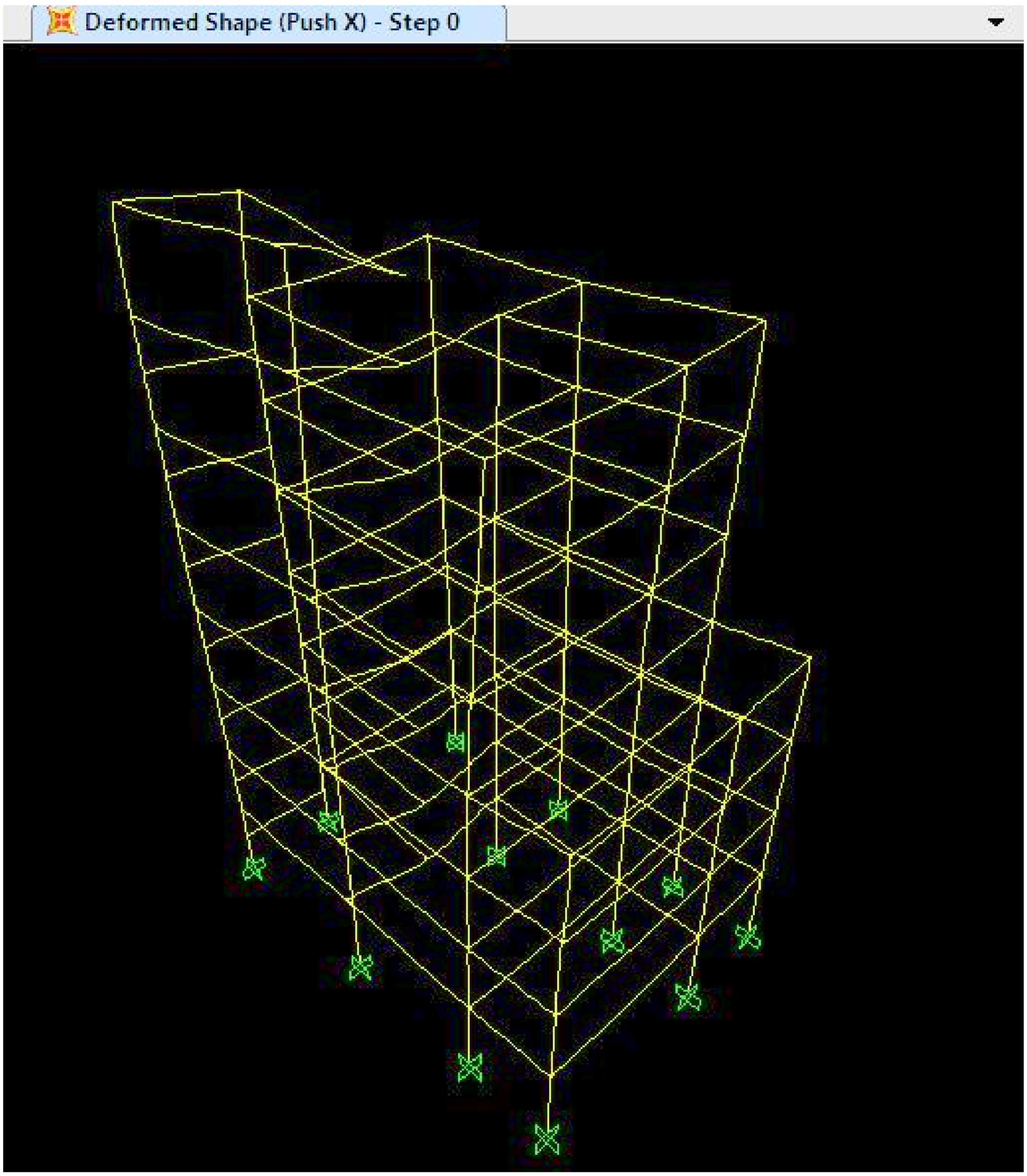

Figure 8-13- Push X deformed shape- Step 0

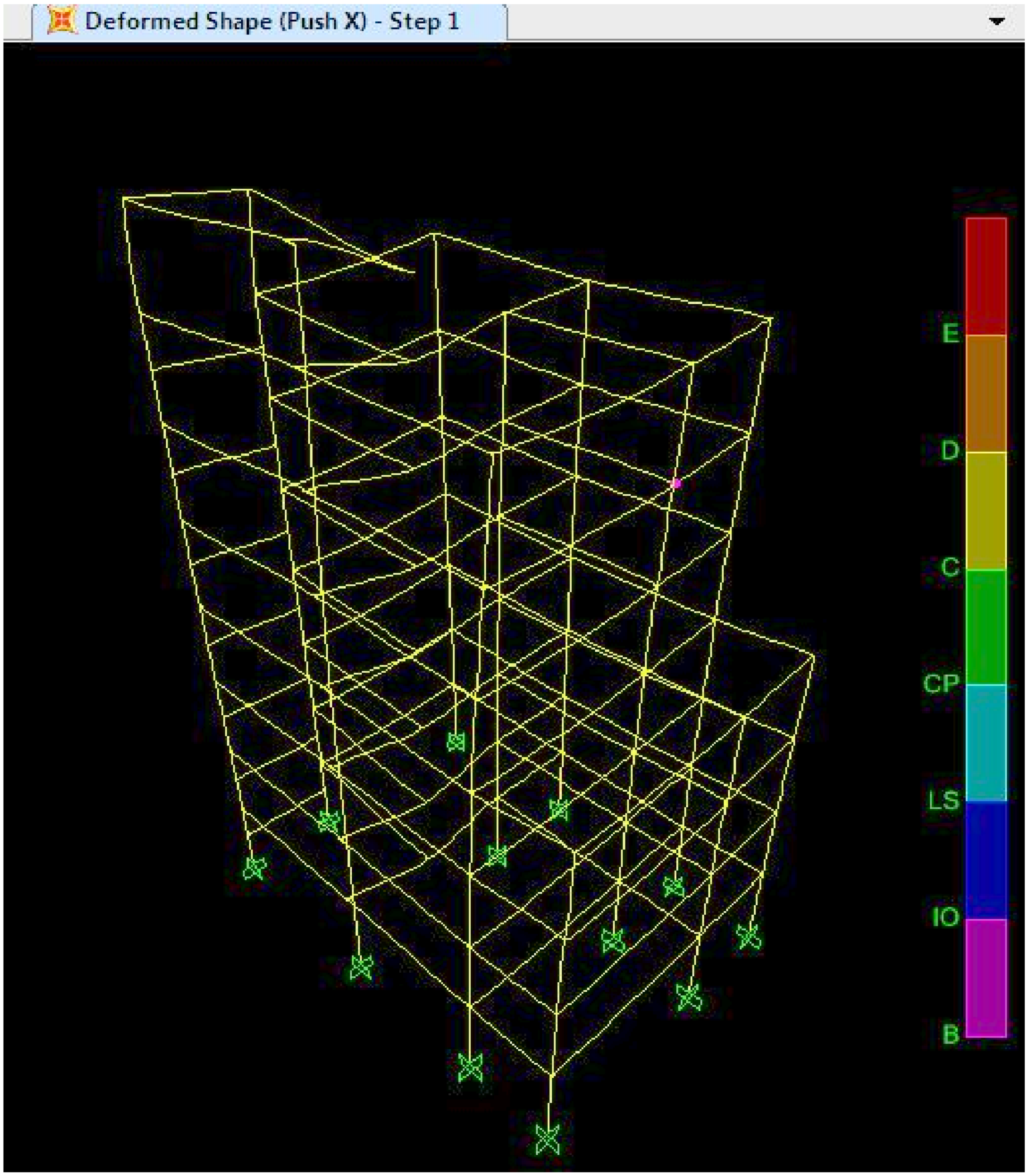

Figure 8-14- Push X deformed shape- Step 1

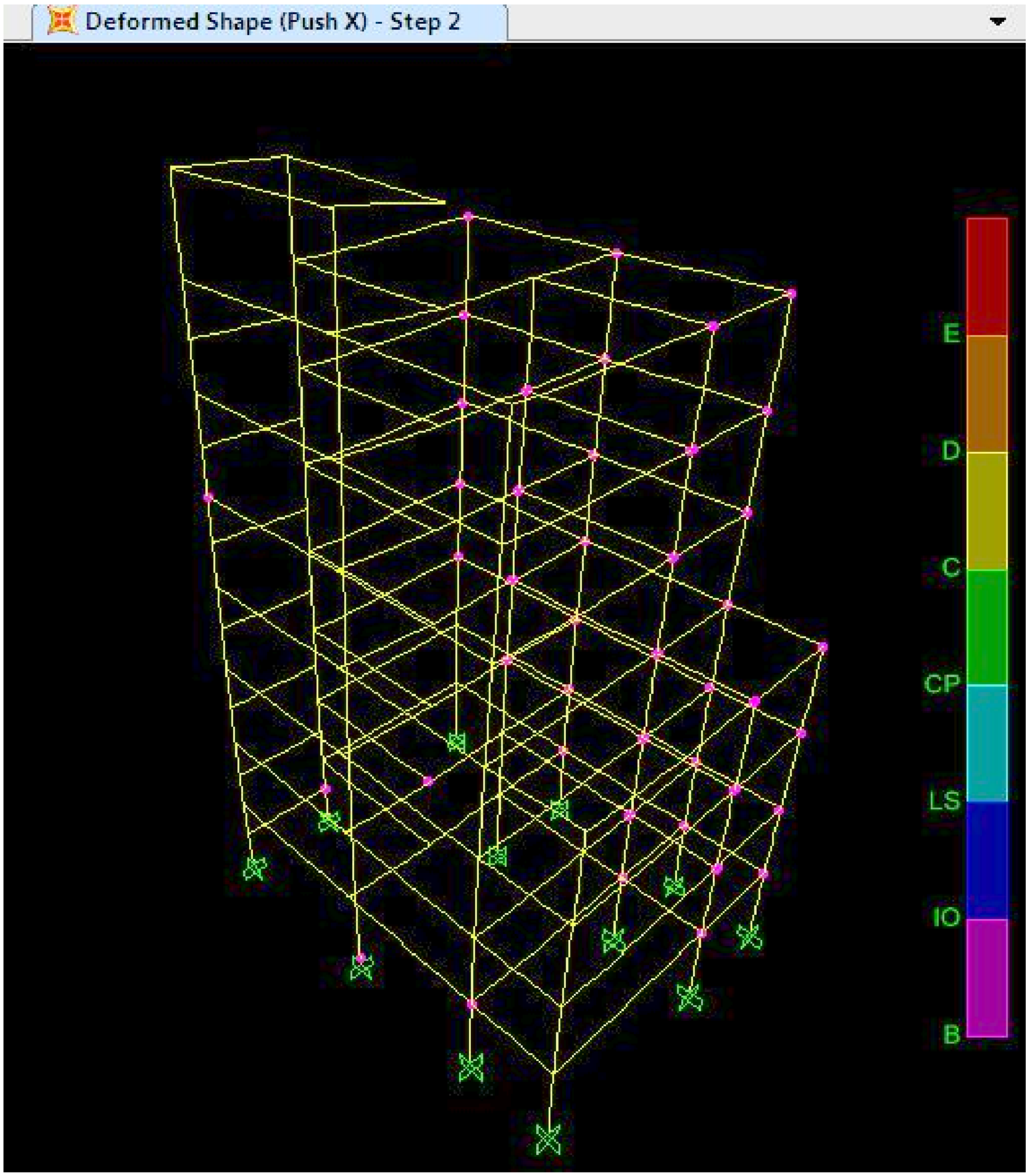

Figure 8-15- Push X deformed shape- Step 2

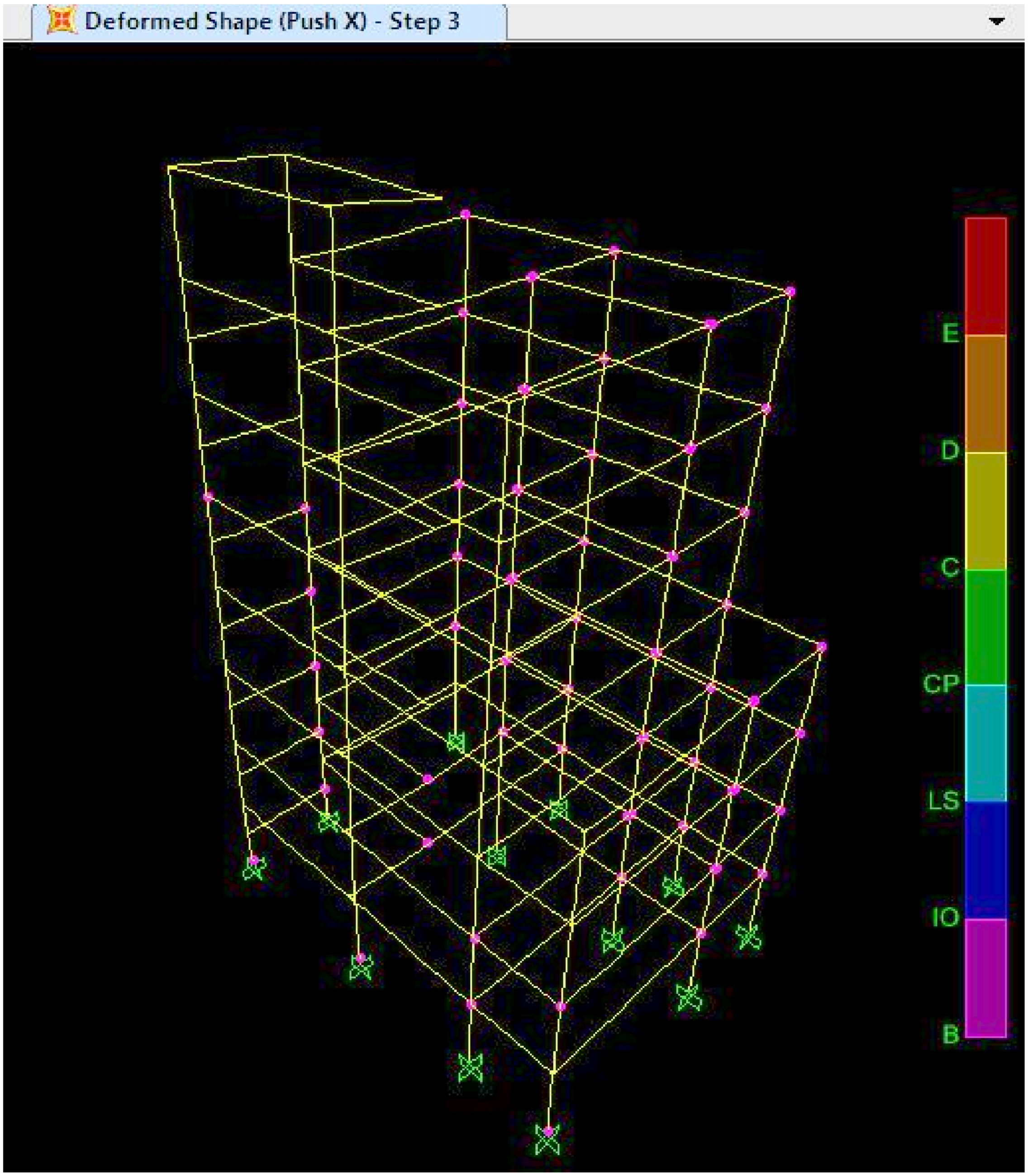

Figure 8-16- Push X deformed shape- Step 3

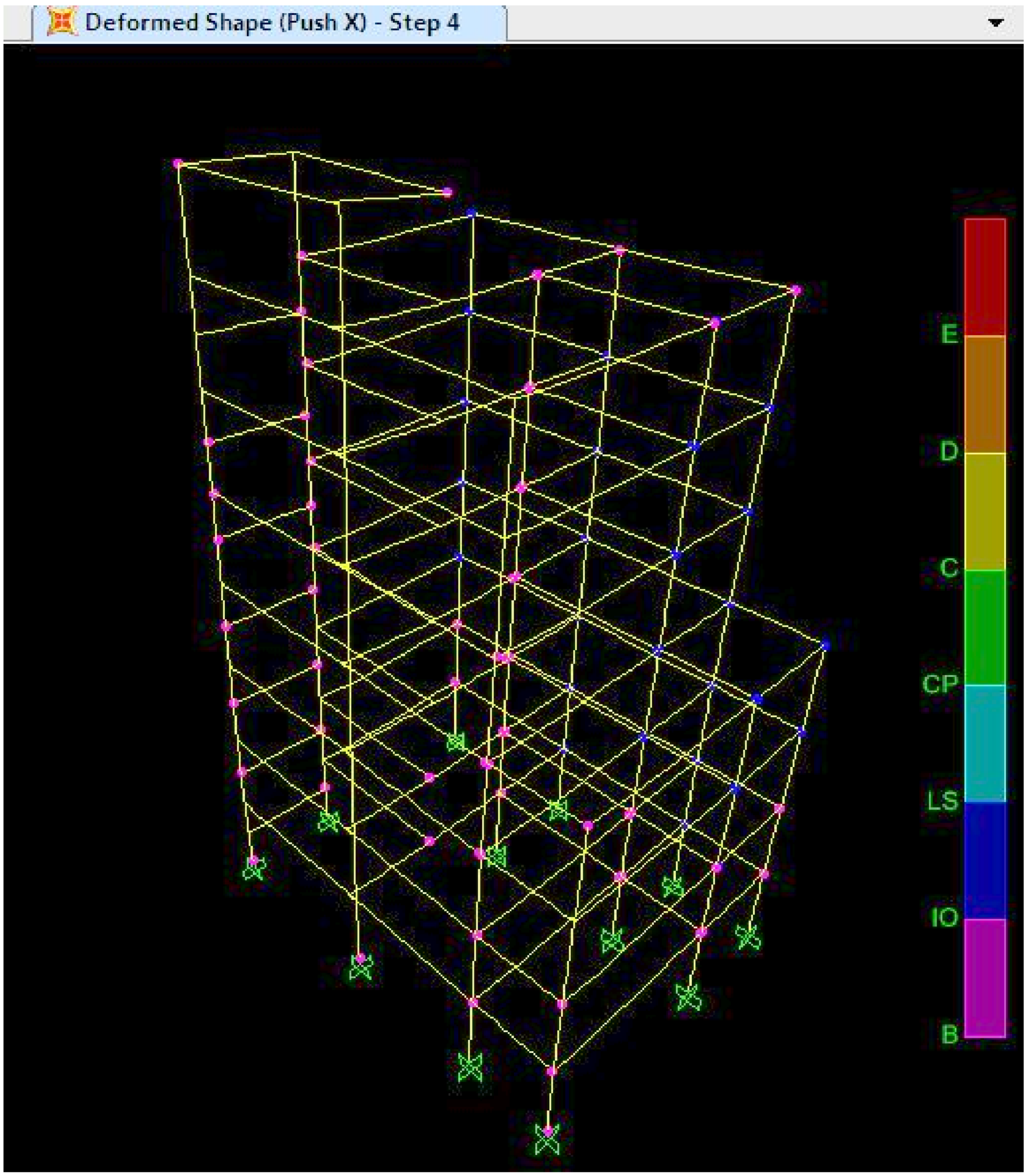

Figure 8-17- Push X deformed shape- Step 4

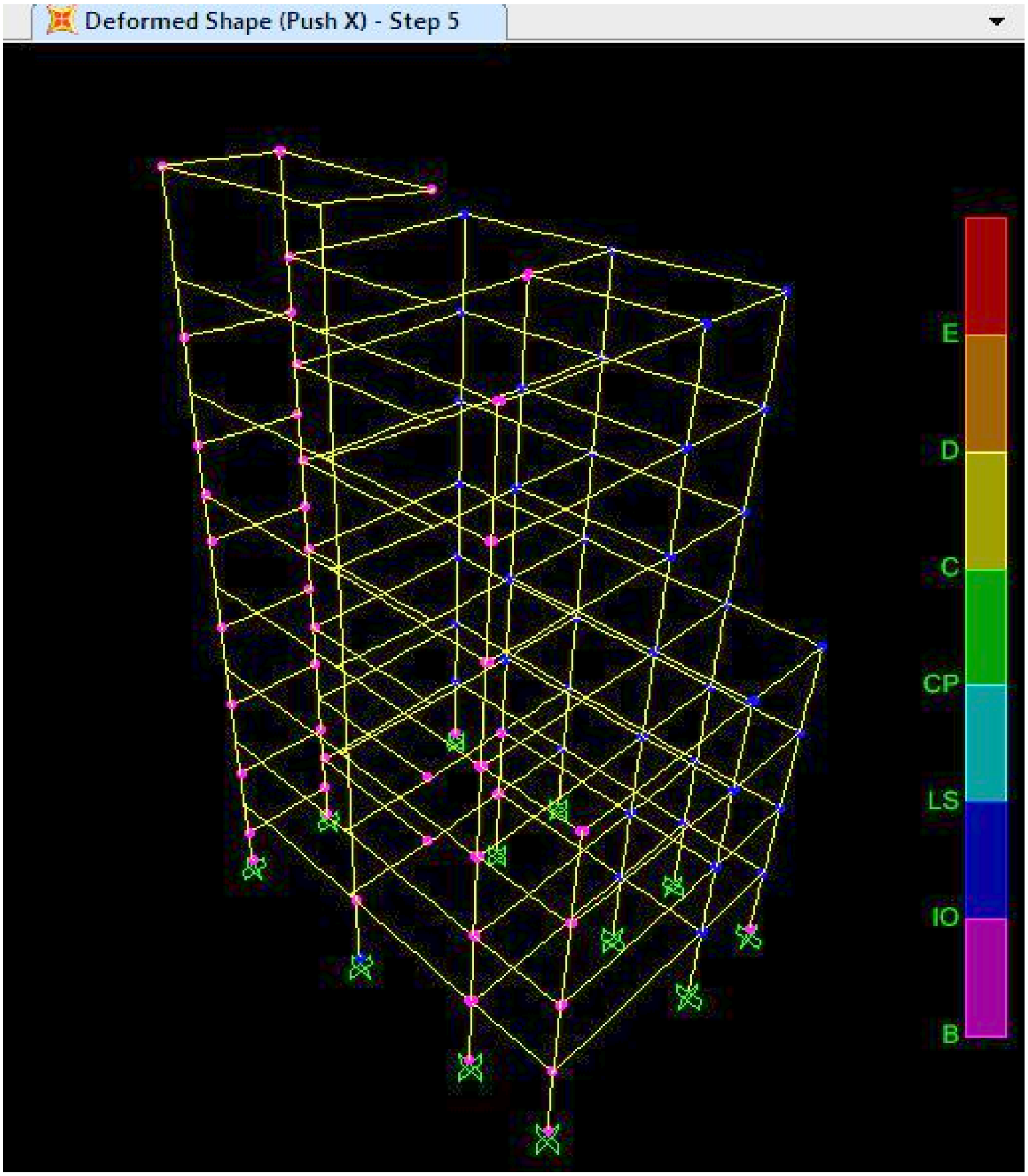

Figure 8-18- Push X deformed shape- Step 5

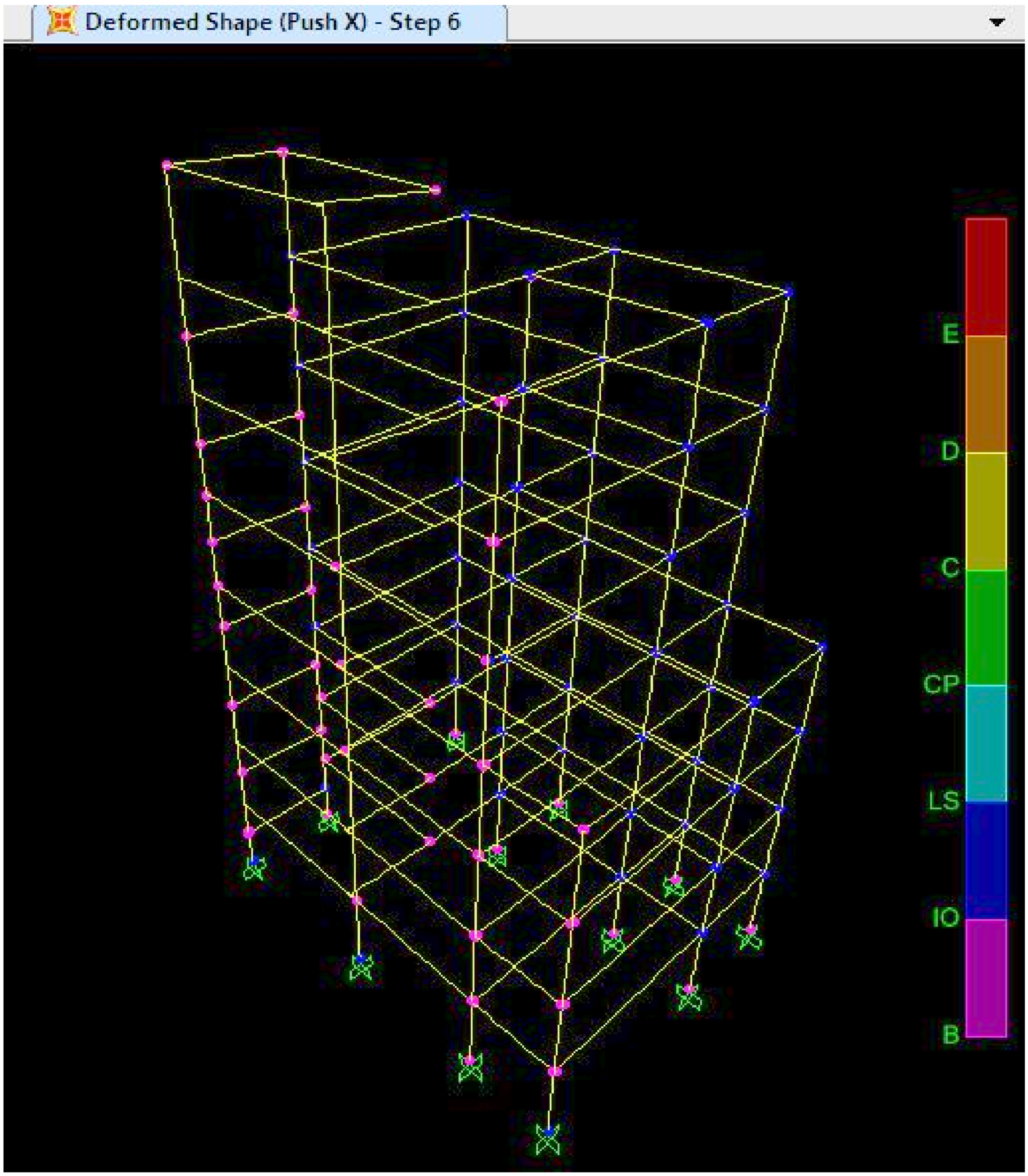

Figure 8-19- Push X deformed shape- Step 6

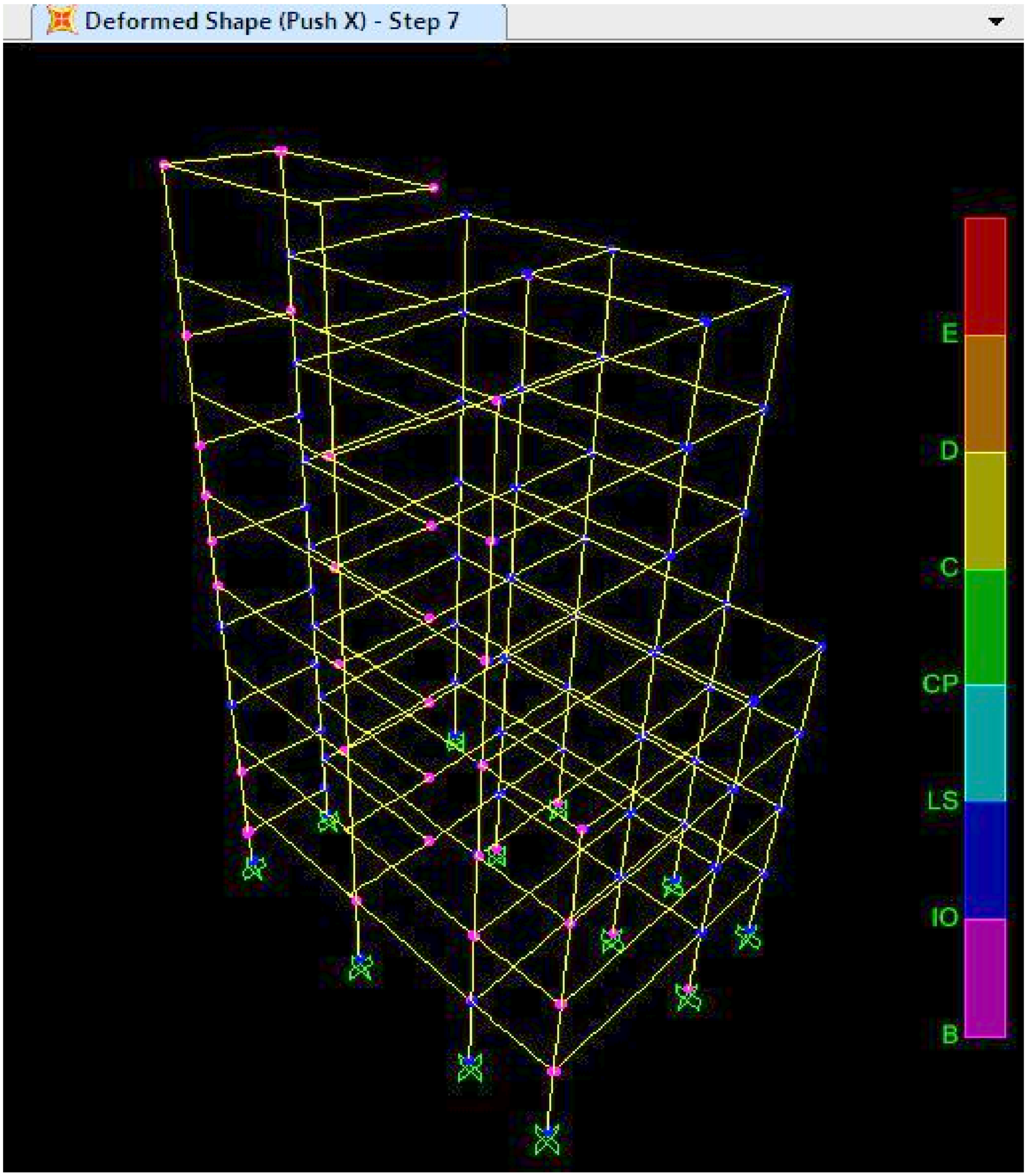

Figure 8-20- Push X deformed shape- Step 7

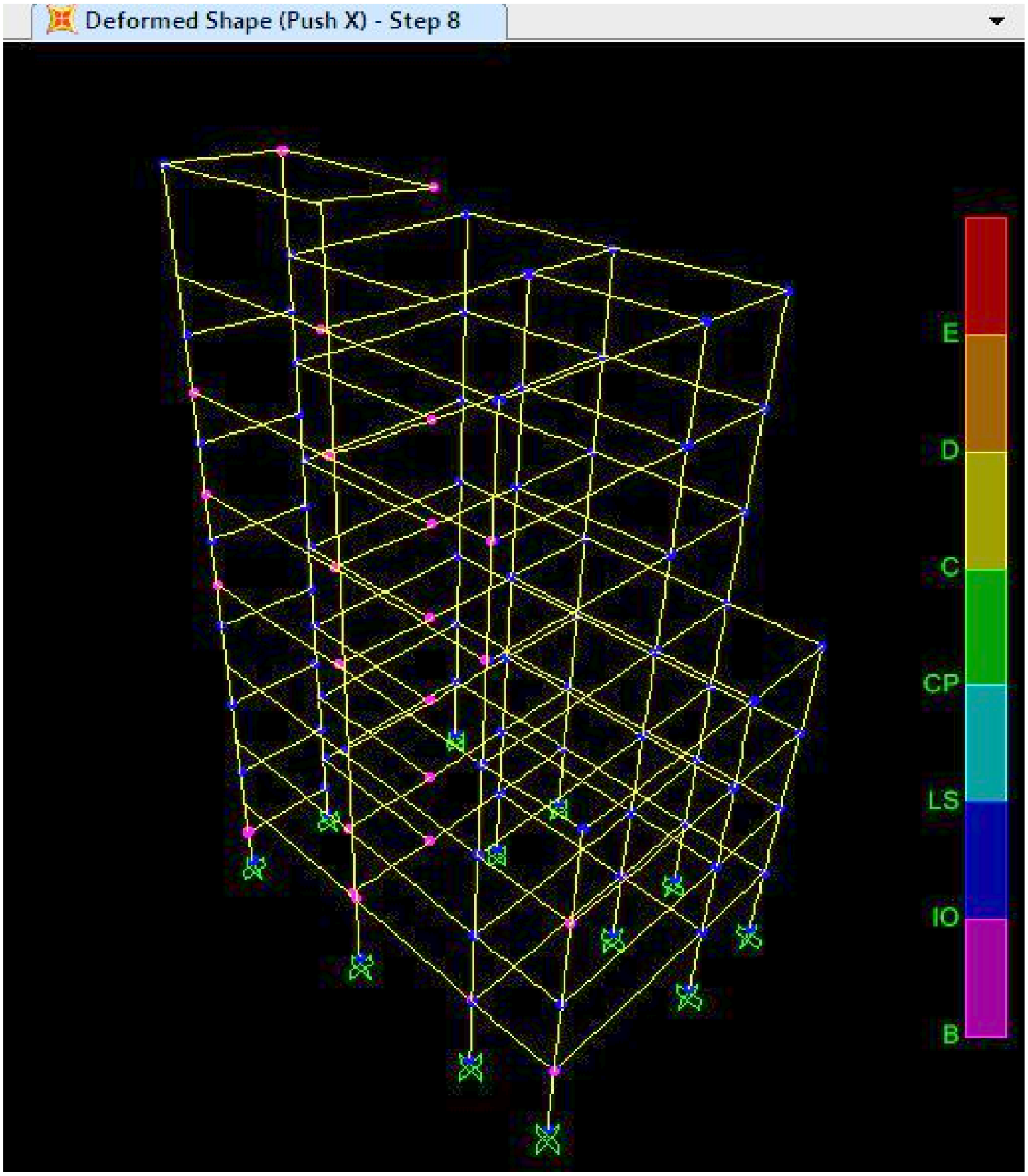

Figure 8-21- Push X deformed shape- Step 8

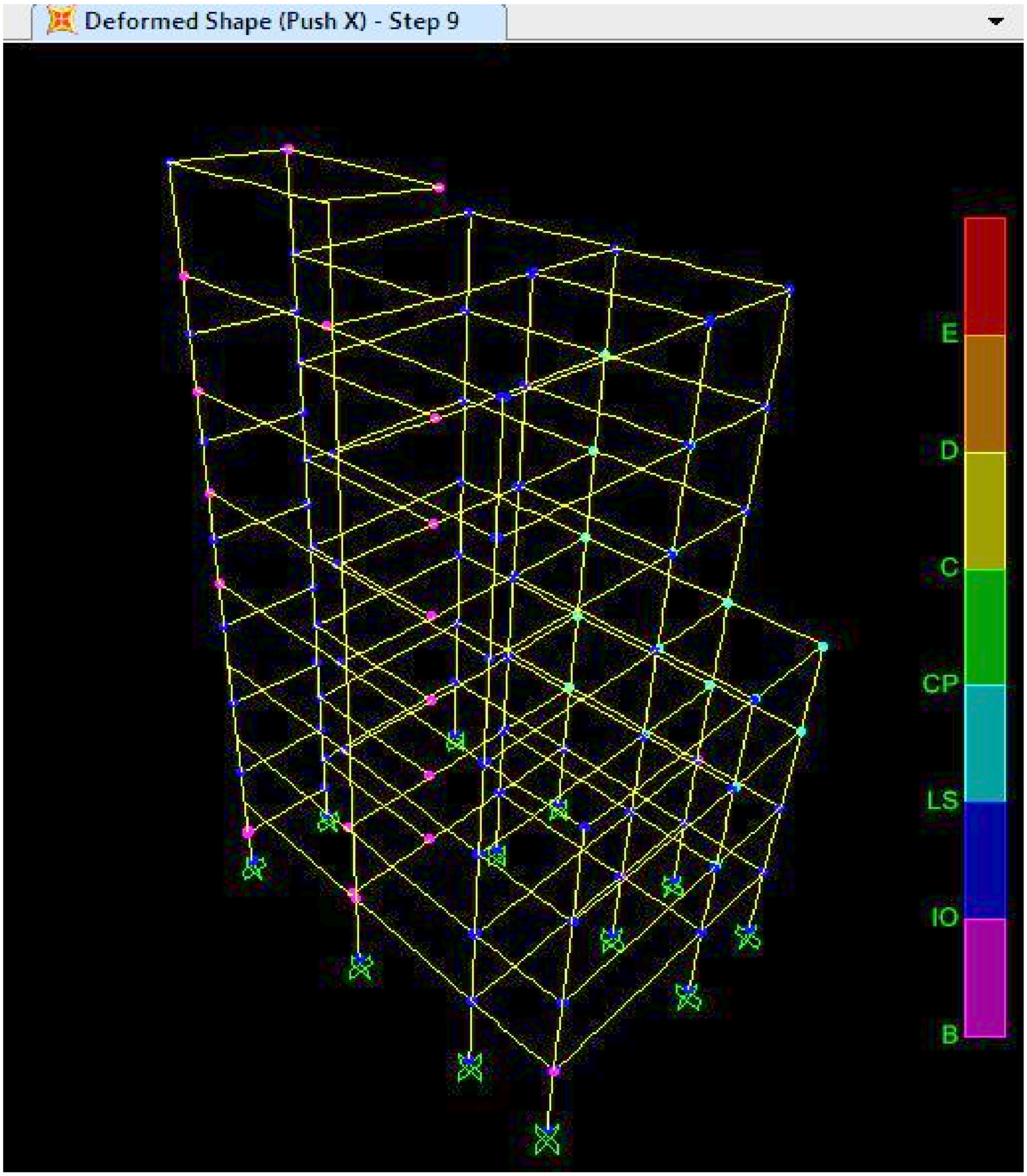

Figure 8-22- Push X deformed shape- Step 9

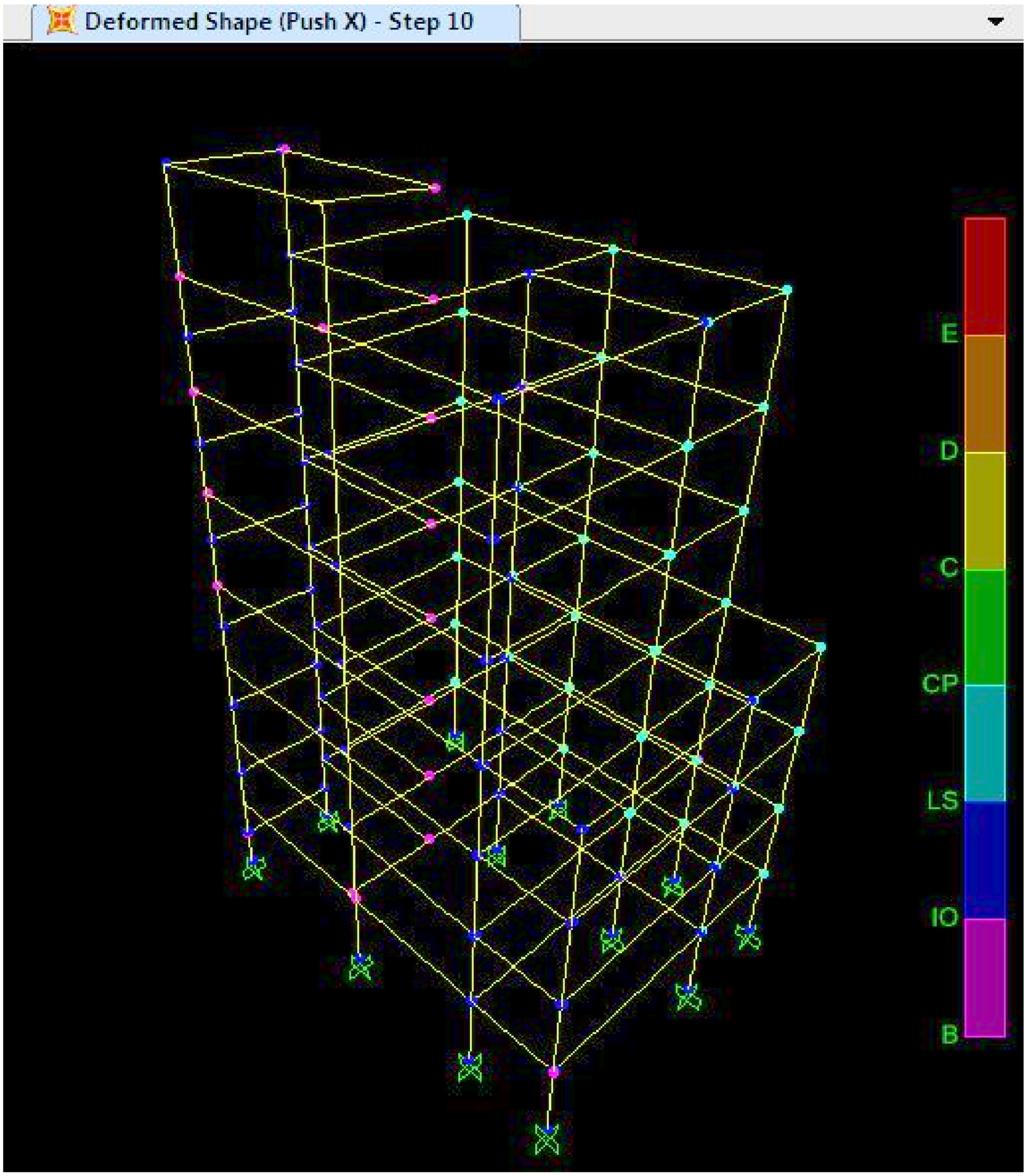

Figure 8-23- Push X deformed shape- Step 10

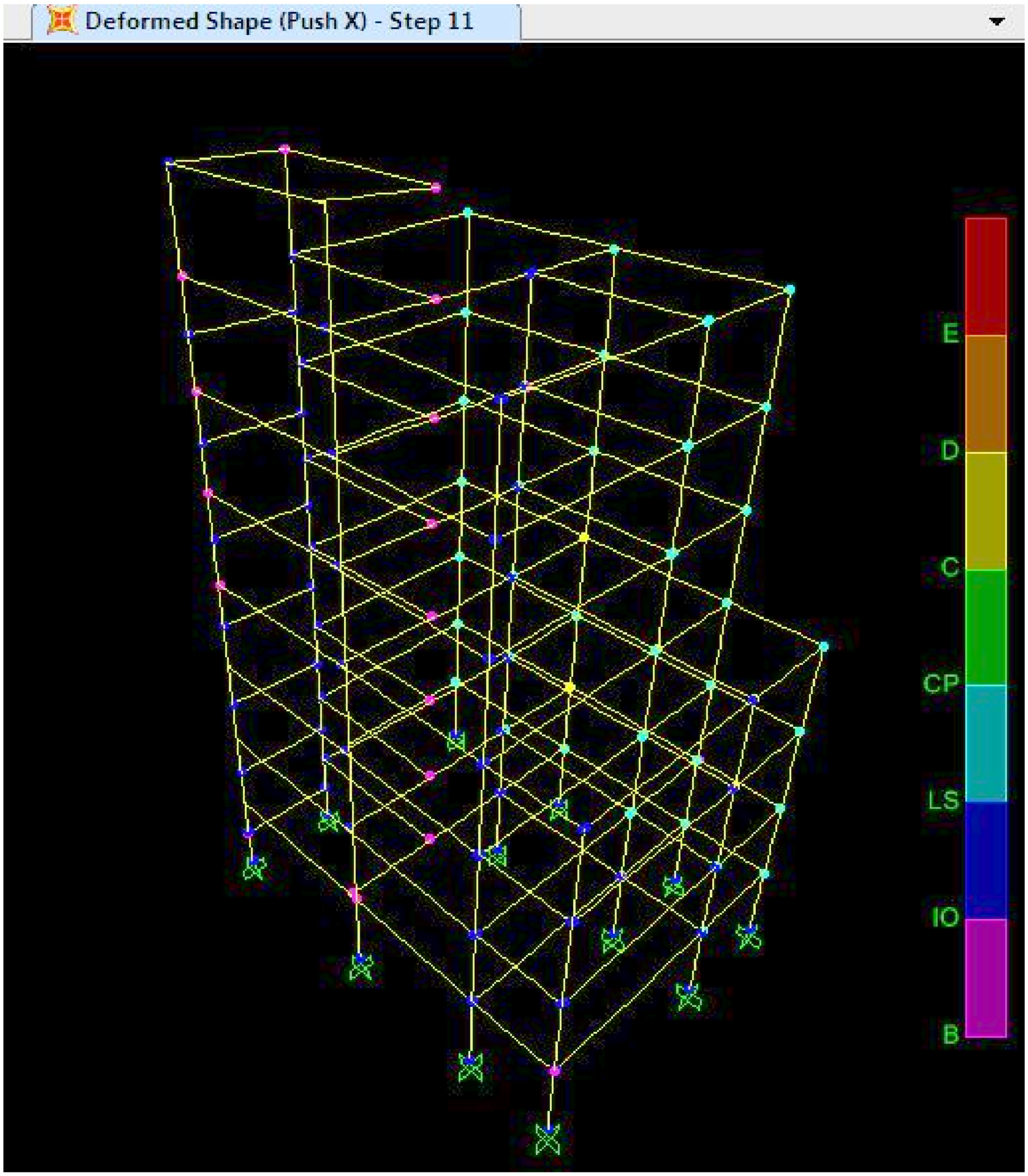

Figure 8-24- Push X deformed shape- Step 11

8-2-1-2) Pushover Curves:

A) Base reaction-displacement:

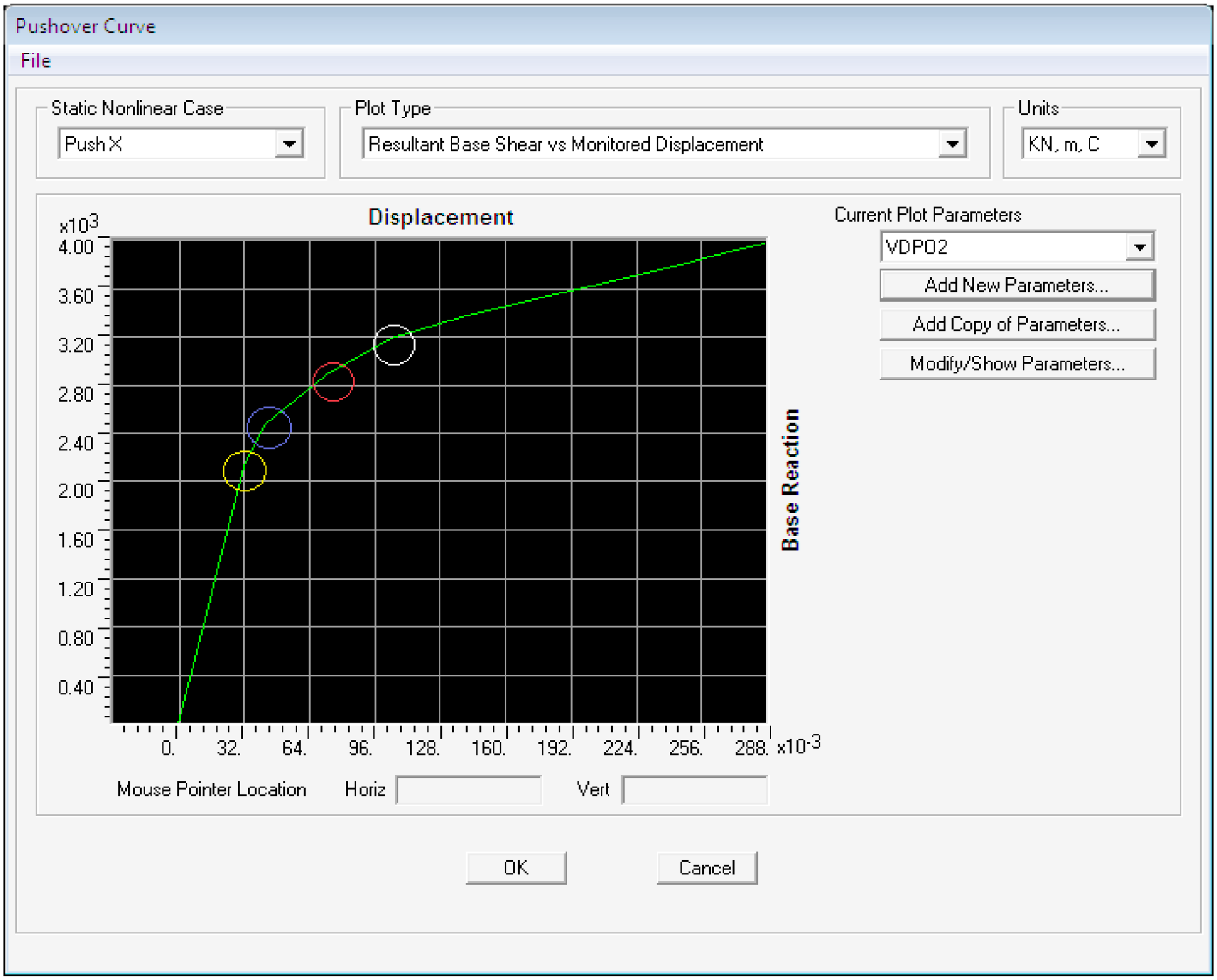

Figure 8-25- Base reaction-displacement Curve in X direction

The points at which we can see a significant change in the slope of the graph are indicated by some circles around them. In fact at these points a major change in the load carrying capacity of some elements is observed as they enter in their plastic regime.

By comparing this capacity curve with the different performance levels specified in chapter 3 (figure 3-6) it becomes evident that this particular structure is within the "Immediate Occupancy" performance range in x direction. This means that minor damage is anticipated in the structural members if the design earthquake happens.

B) ATC-40 capacity spectrum:

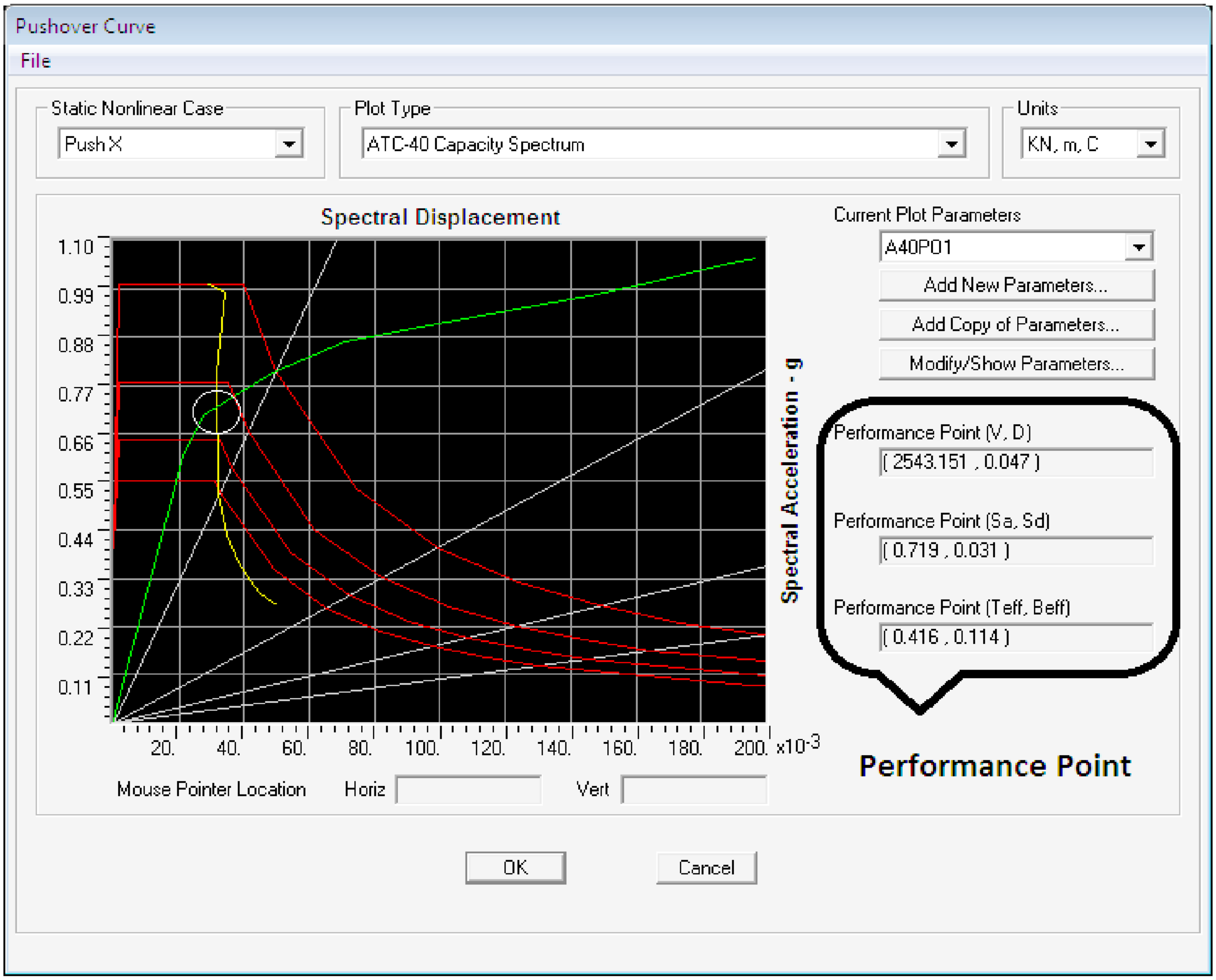

Figure 8-26- ATC-40 capacity spectrum Curve in X direction

The point of intersection of the demand curve and the capacity curve that is the performance point is shown with a circle around it. As it is shown in the right hand side of the graph, the coordinate of this point is: $S_a = 0.719, S_d = 0.031 \; or \; T_{eff} = 0.416 \, , B_{eff} = 0.114$

8-2-1-3) Tabular Data:

A) Pushover curve tabular data:

TABLE: Pushover Curve - Push X											
Step	Displacement	BaseForce	AtoB	BtoIO	IOtoLS	LStoCP	CPtoC	CtoD	DtoE	BeyondE	Total
	m	KN									
0	-0.000205	0	400	0	0	0	0	0	0	0	400
1	0.000799	90.117	399	1	0	0	0	0	0	0	400
2	0.031473	2124.632	344	56	0	0	0	0	0	0	400
3	0.041242	2465.572	331	69	0	0	0	0	0	0	400
4	0.071901	2894.585	305	62	33	0	0	0	0	0	400
5	0.103213	3182.741	278	65	57	0	0	0	0	0	400
6	0.138184	3358.965	263	66	71	0	0	0	0	0	400
7	0.177056	3521.072	260	44	96	0	0	0	0	0	400
8	0.212676	3655.95	252	31	117	0	0	0	0	0	400
9	0.242676	3779.13	250	25	109	16	0	0	0	0	400
10	0.275689	3929.237	248	24	83	45	0	0	0	0	400
11	0.287223	3970.273	248	23	82	42	0	5	0	0	400

Table 8-1- Pushover curve tabular data in X direction

B) Pushover curve demand capacity-ATC 40 tabular data:

TABLE: Pushover Curve Demand Capacity - ATC40 - Push X								
Step	Teff	Beff	SdCapacity	SaCapacity	SdDemand	SaDemand	Alpha	PFPhi
			m		m			
0	0.342606	0.05	0	0	0.029158	1	1	1
1	0.342606	0.05	0.00074	0.025379	0.029158	1	0.703929	1.355719
2	0.374978	0.052942	0.021241	0.608137	0.034215	0.979574	0.692583	1.491356
3	0.398111	0.093671	0.027642	0.702106	0.031361	0.796557	0.696154	1.499411
4	0.496713	0.206315	0.048821	0.796592	0.031976	0.521735	0.720345	1.476942
5	0.571639	0.236527	0.070577	0.869477	0.03487	0.429589	0.72566	1.465316
6	0.648983	0.259163	0.0943	0.901329	0.038124	0.364394	0.738775	1.467533
7	0.720737	0.268713	0.121072	0.938271	0.041695	0.323126	0.743938	1.4641
8	0.776496	0.272059	0.145632	0.972341	0.044684	0.298339	0.74537	1.461772
9	0.815945	0.271659	0.166344	1.005833	0.046983	0.284094	0.744828	1.460109
10	0.852334	0.268856	0.189129	1.048042	0.049297	0.273174	0.743224	1.458756
11	0.865438	0.268754	0.197147	1.059636	0.050063	0.269082	0.742769	1.457937

Table 8-2- Pushover curve demand capacity-ATC 40 tabular data in X direction

8-2-2) Y direction:

8-2-2-1) Hinges:

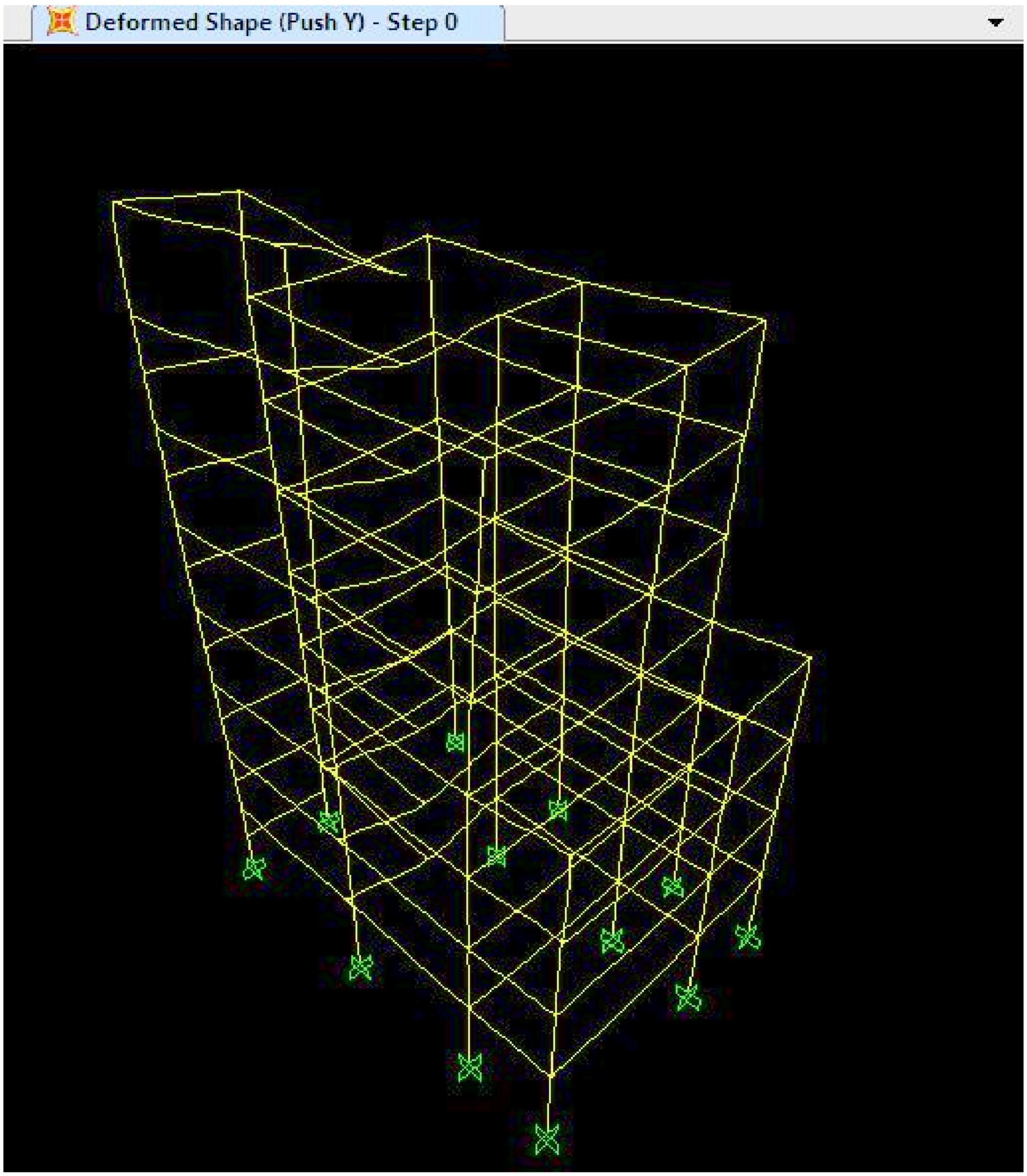

Figure 8-27- Push Y deformed shape- Step 0

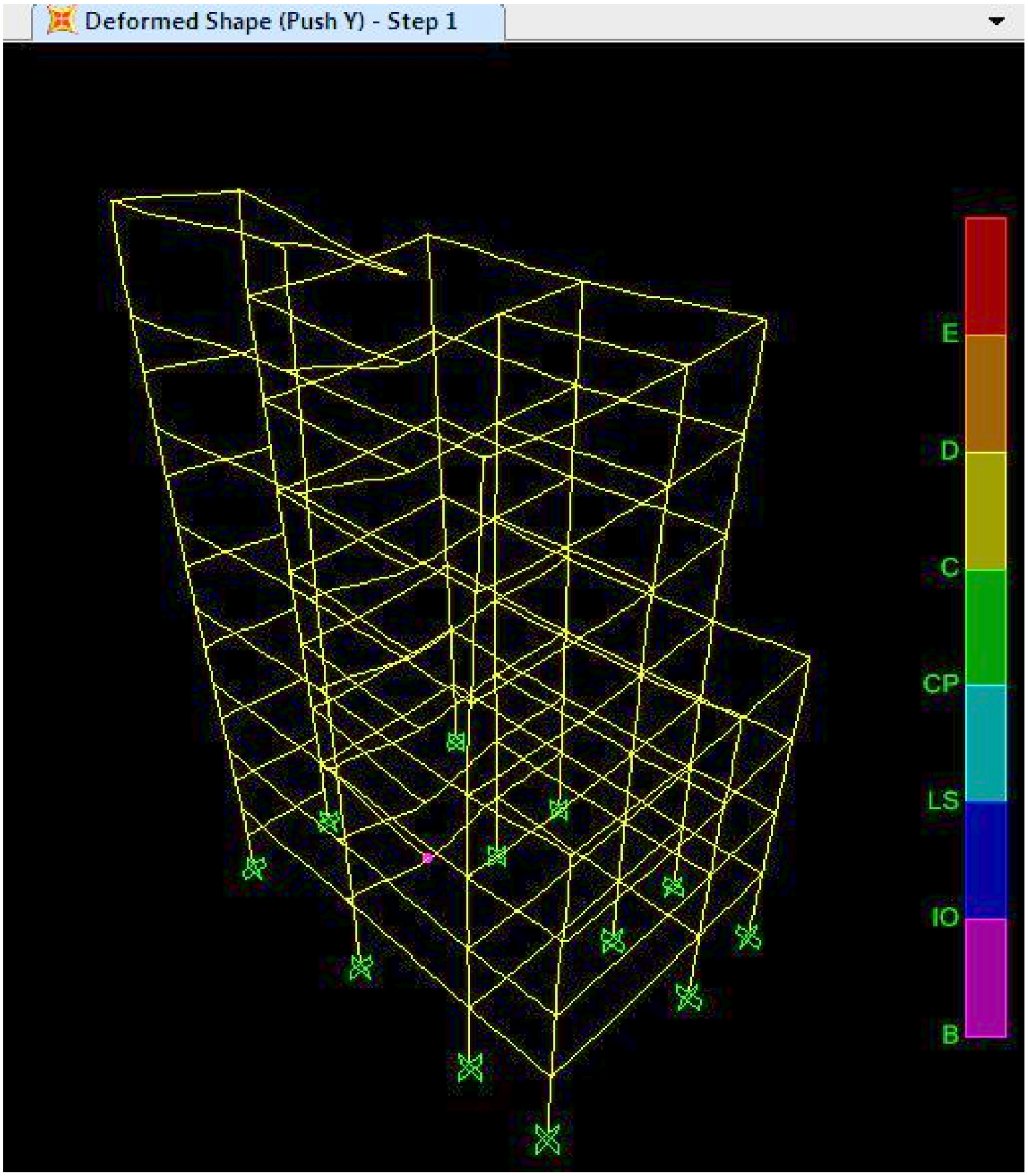

Figure 8-28- Push Y deformed shape- Step 1

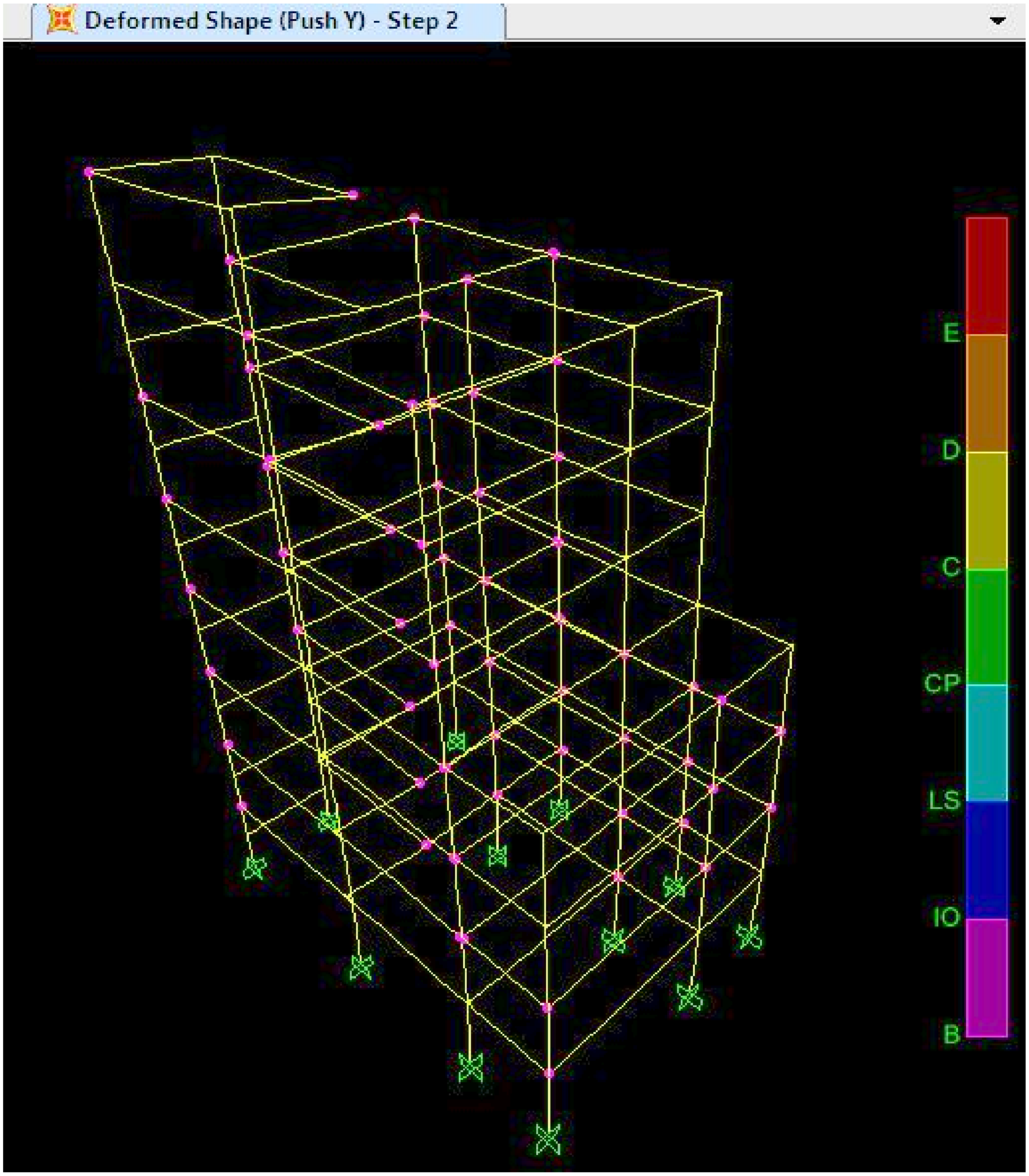

Figure 8-29- Push Y deformed shape- Step 2

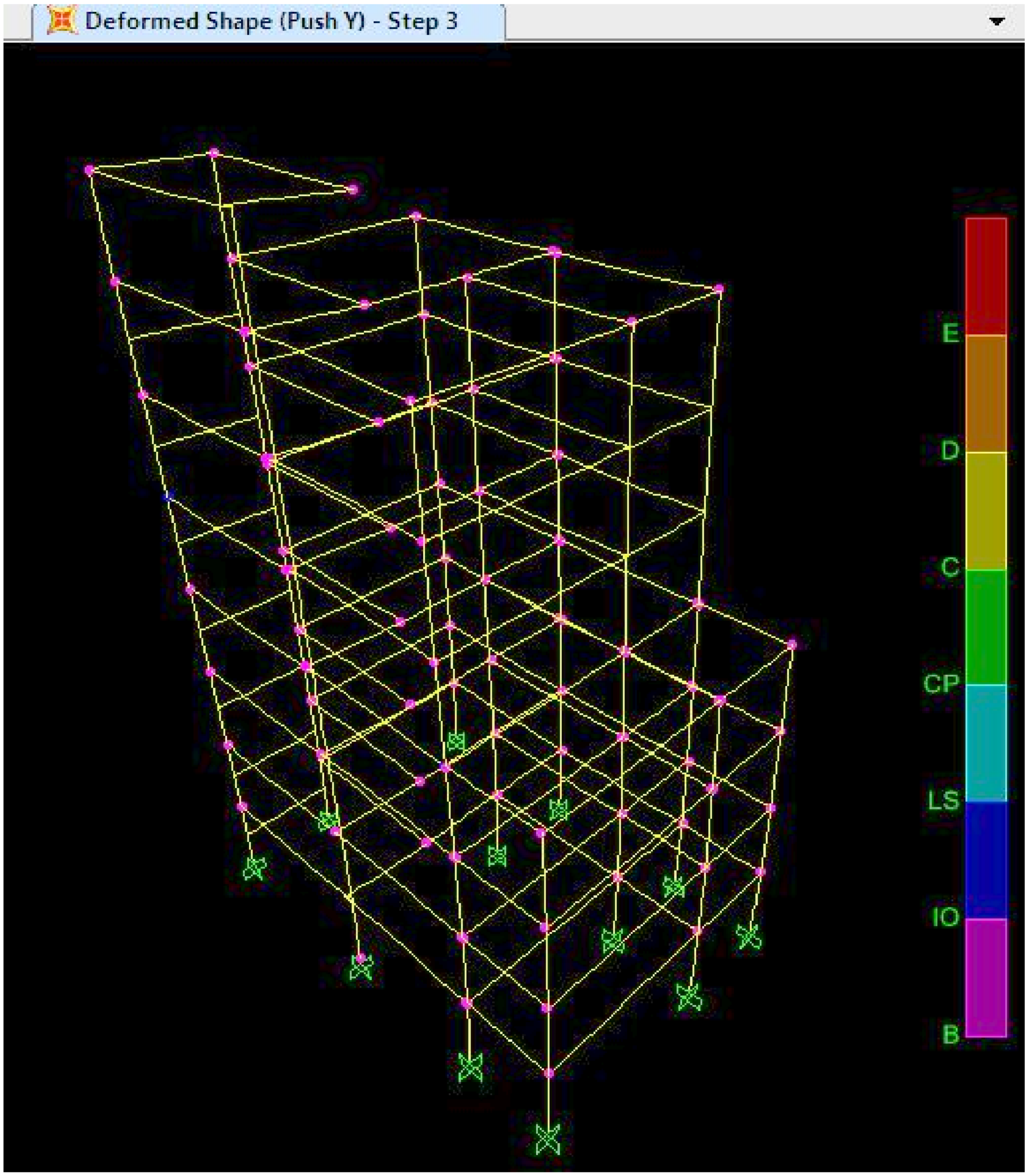

Figure 8-30- Push Y deformed shape- Step 3

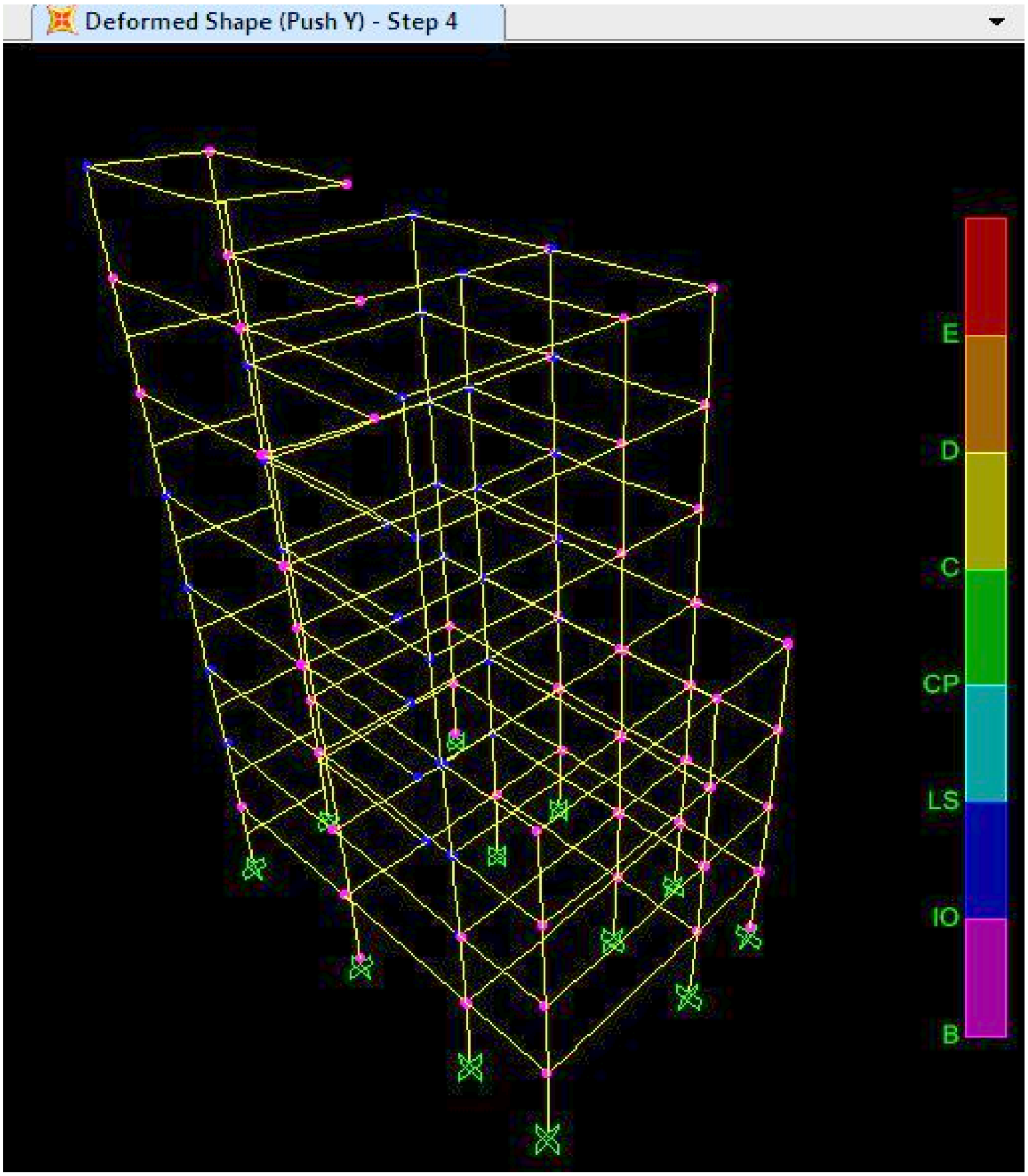

Figure 8-31- Push Y deformed shape- Step 4

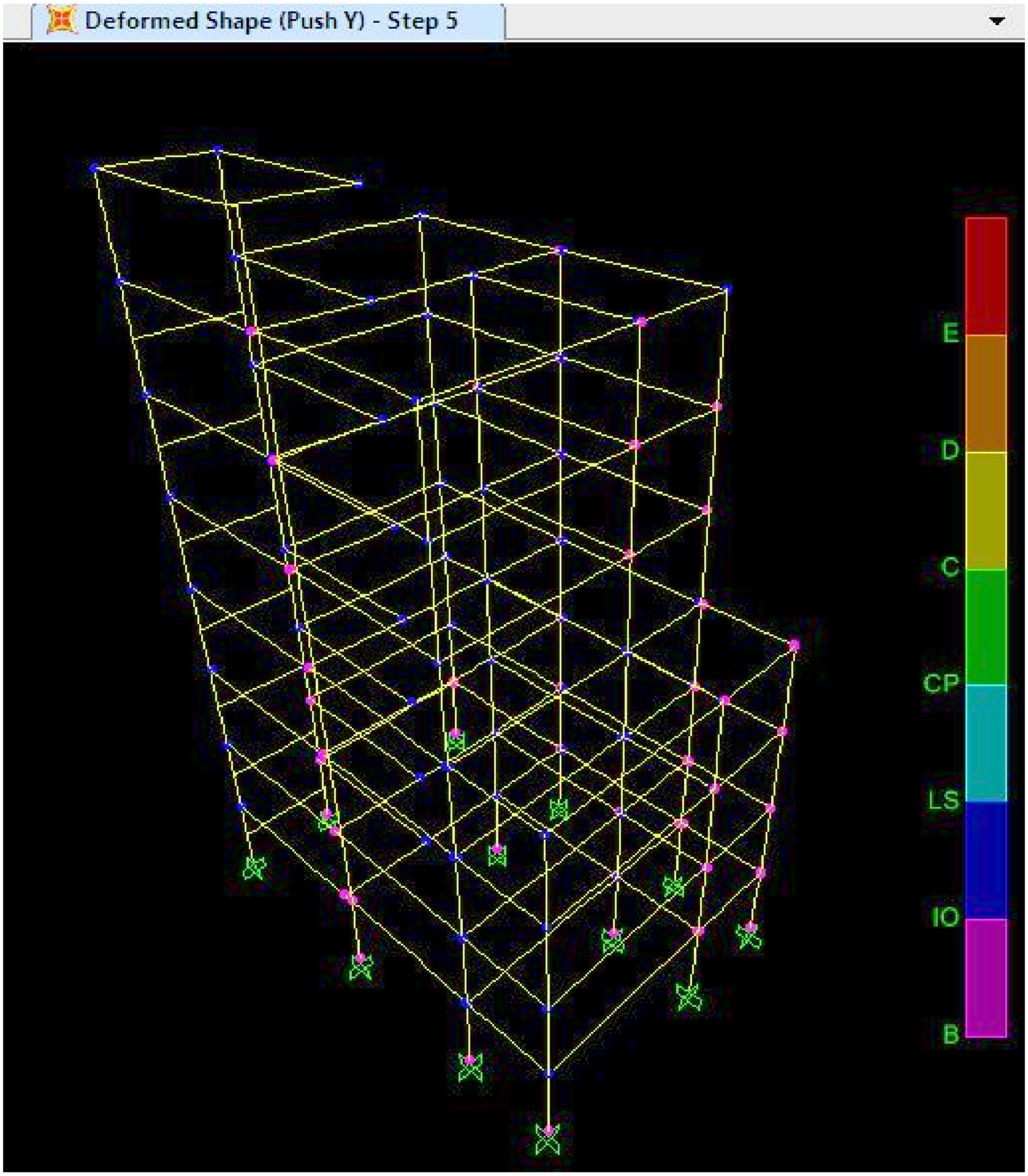

Figure 8-32- Push Y deformed shape- Step 5

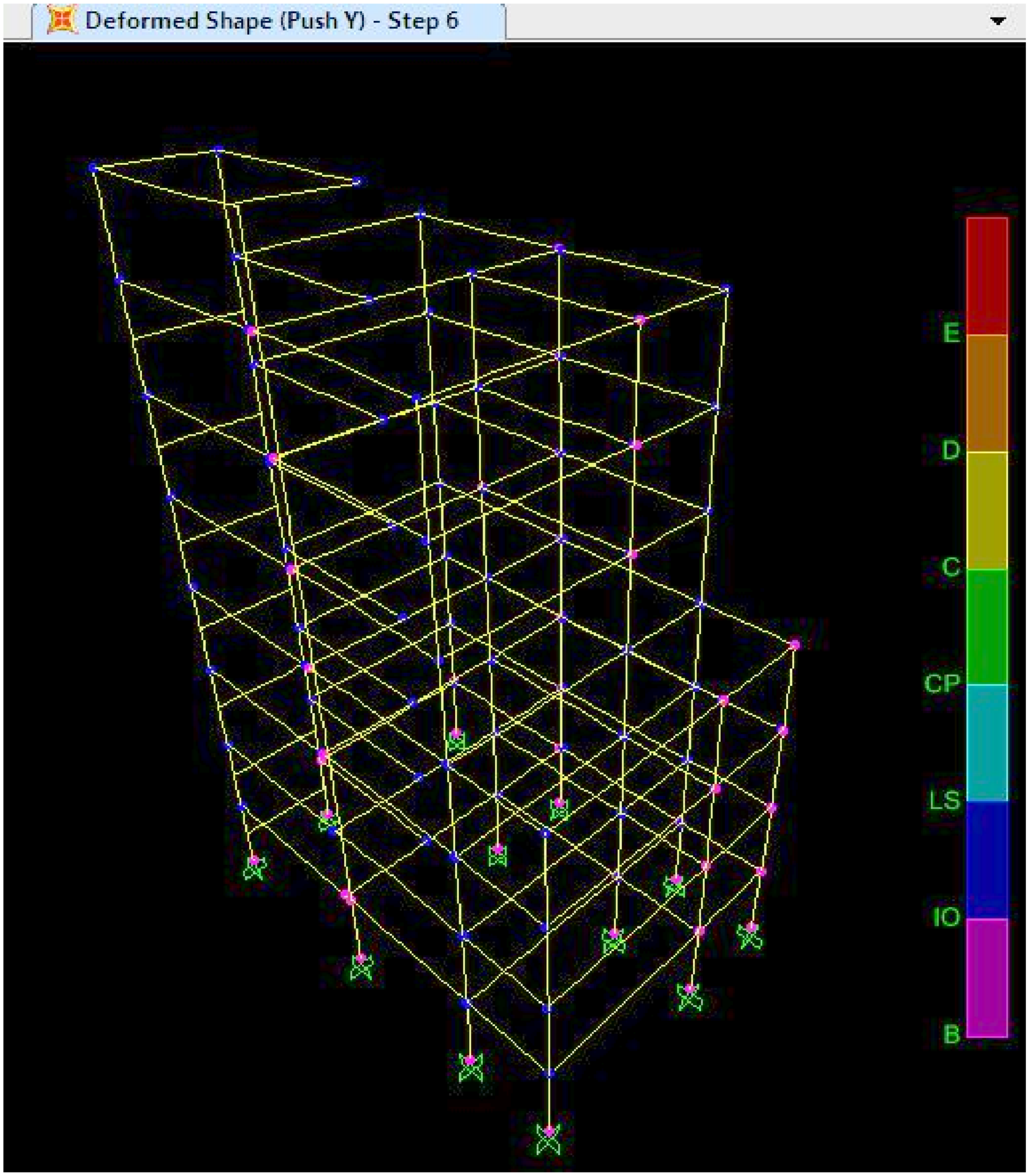

Figure 8-33- Push Y deformed shape- Step 6

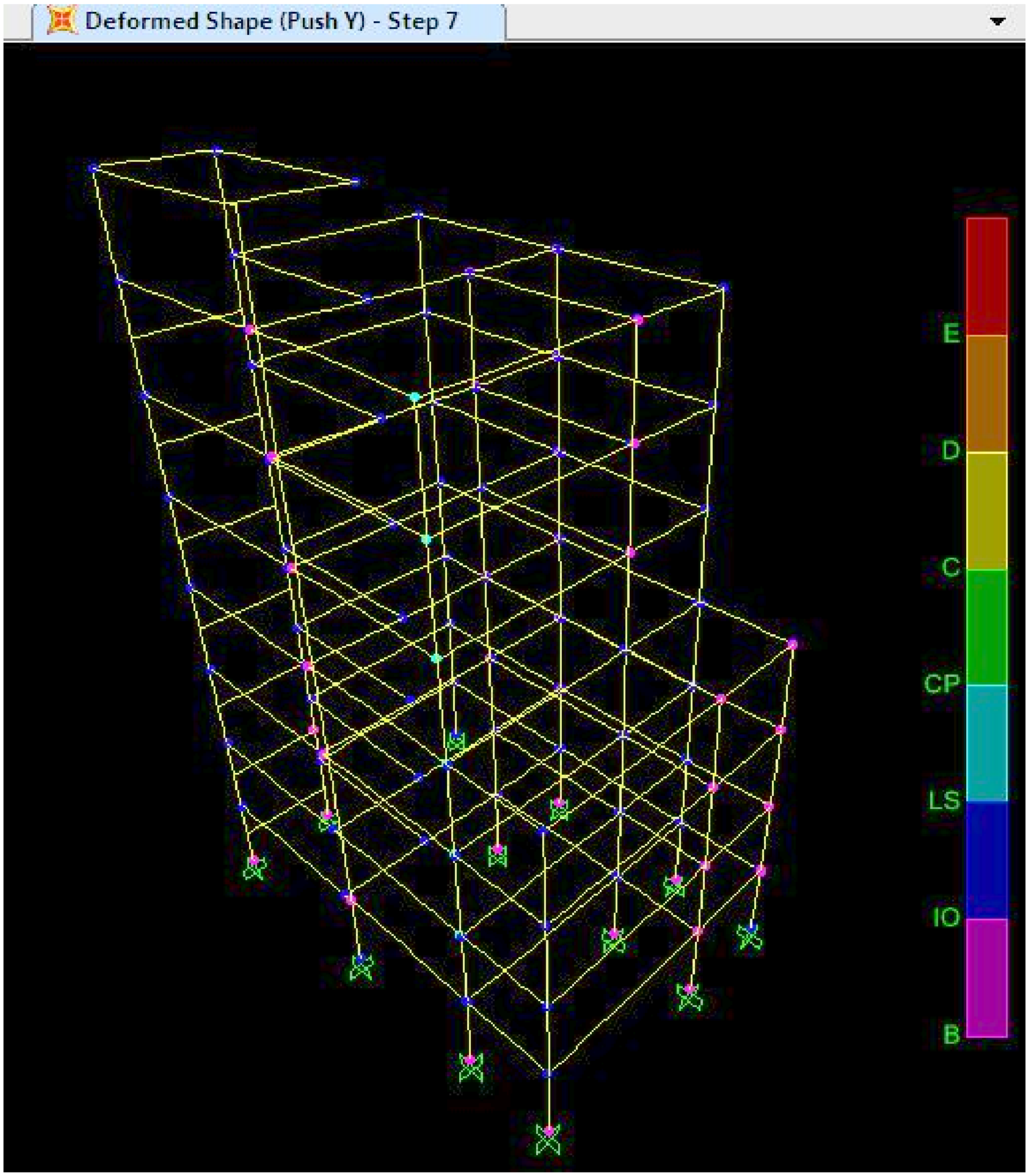

Figure 8-34- Push Y deformed shape- Step 7

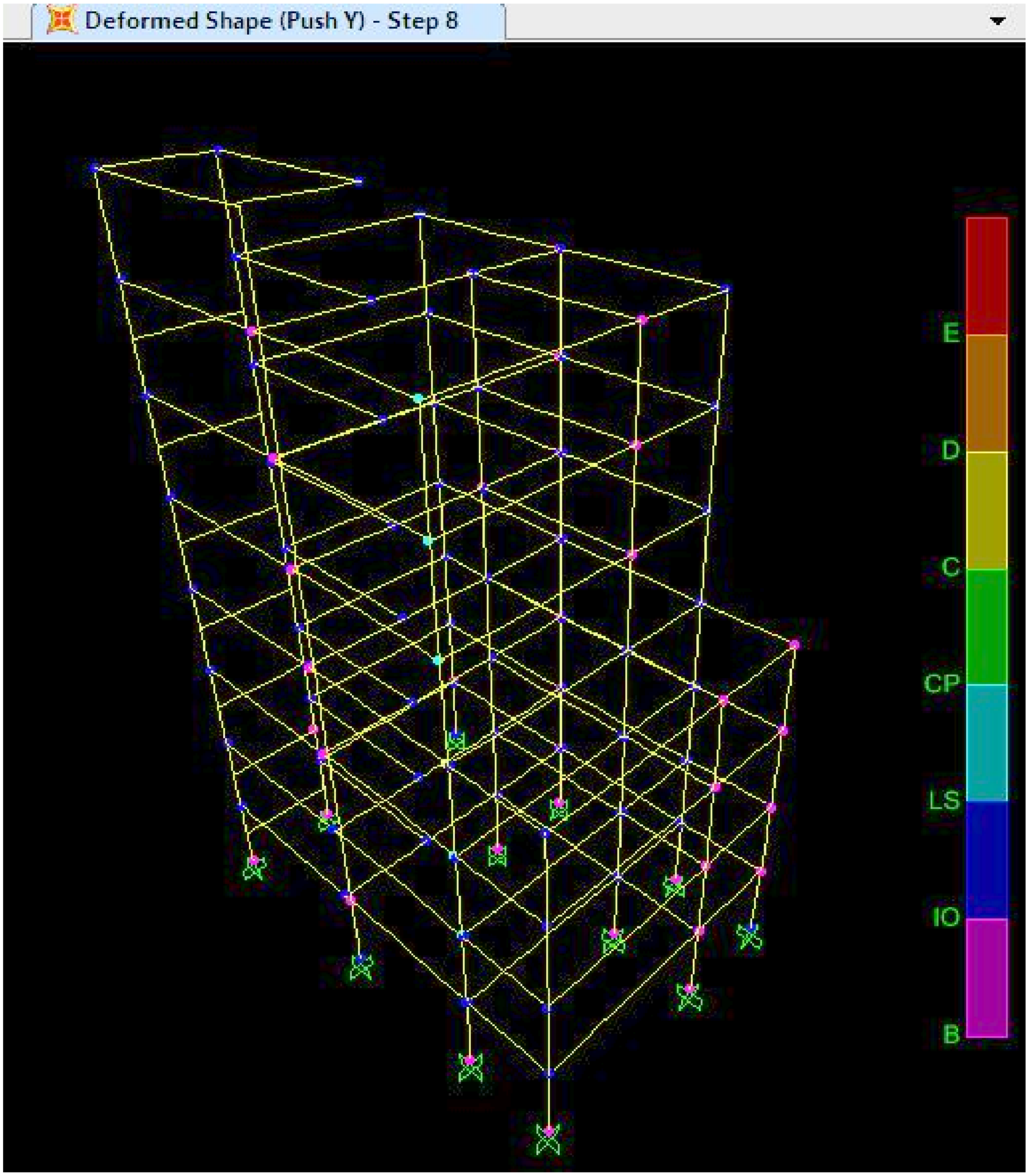

Figure 8-35- Push Y deformed shape- Step 8

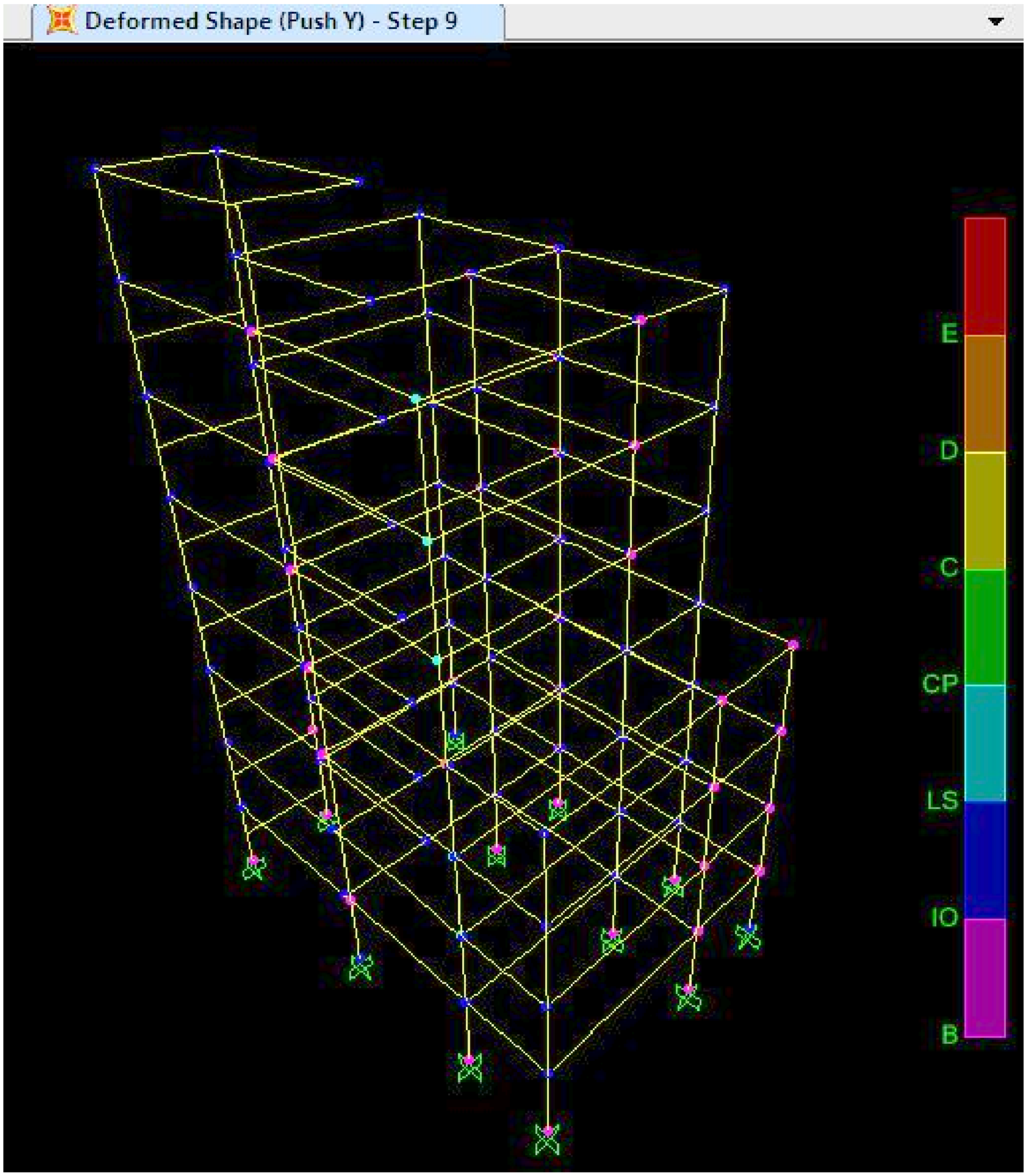

Figure 8-36- Push Y deformed shape- Step 9

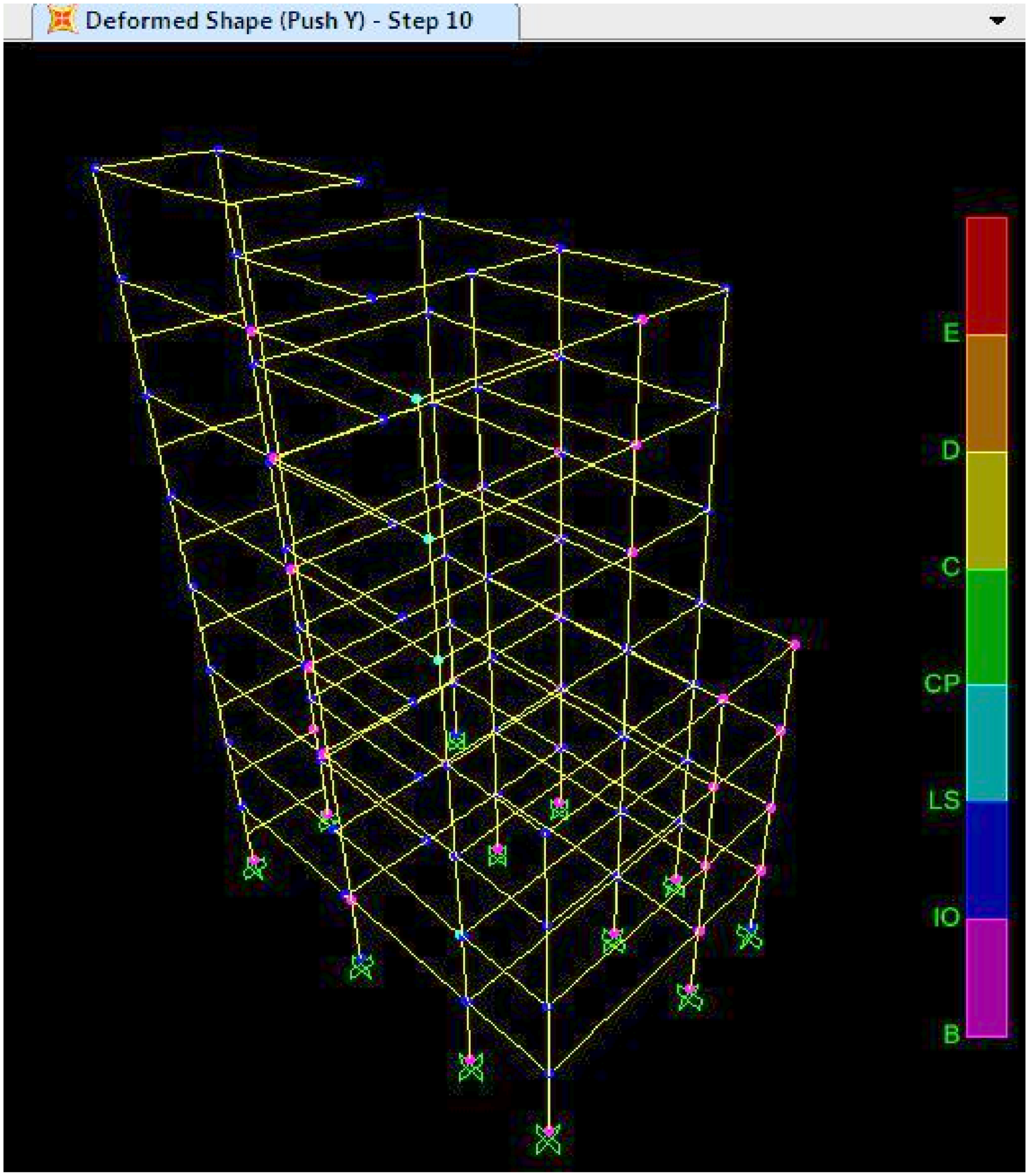

Figure 8-37- Push Y deformed shape- Step 10

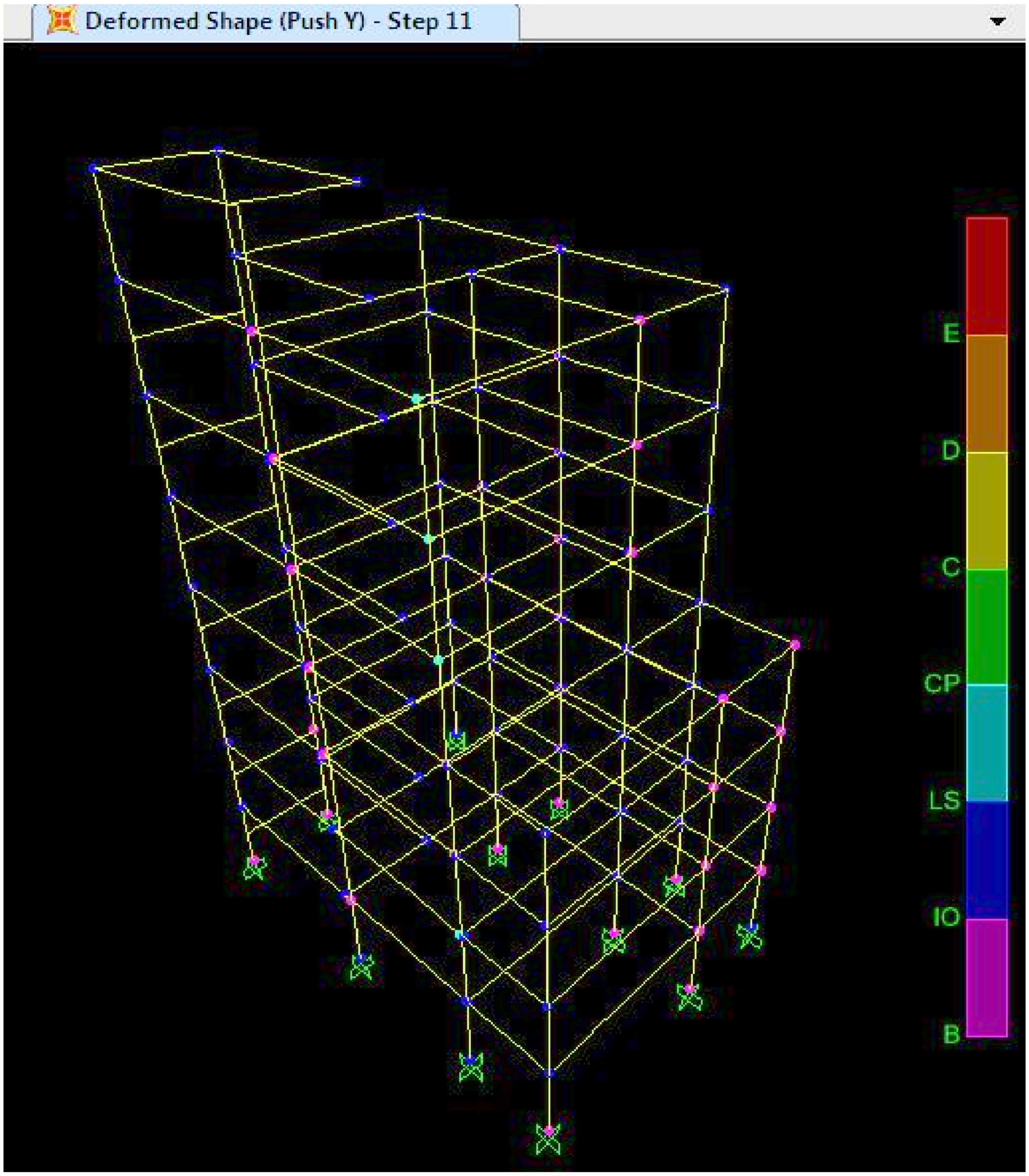

Figure 8-38- Push Y deformed shape- Step 11

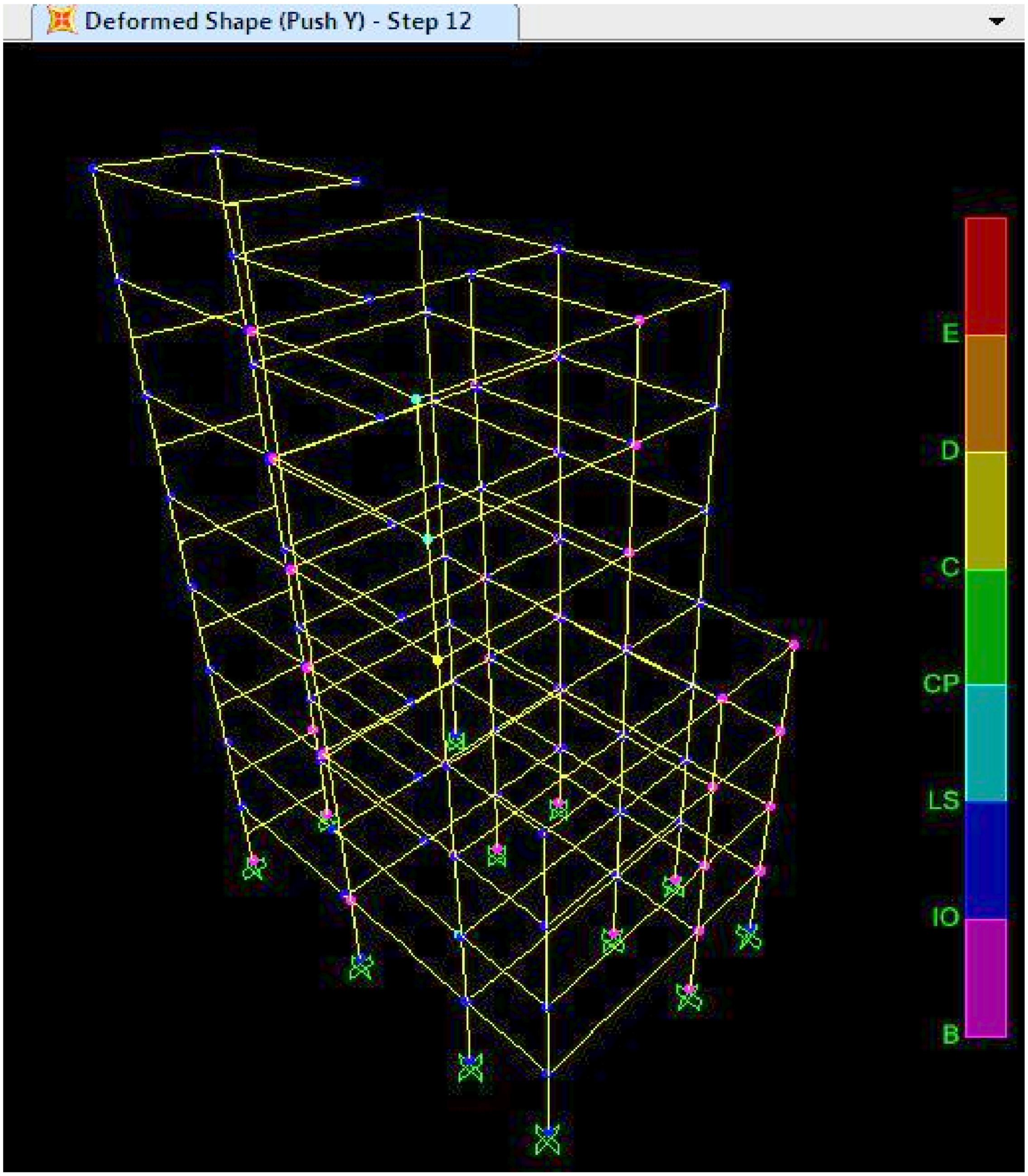

Figure 8-39- Push Y deformed shape- Step 12

8-2-2-2) Pushover Curves:

A) Base reaction-displacement:

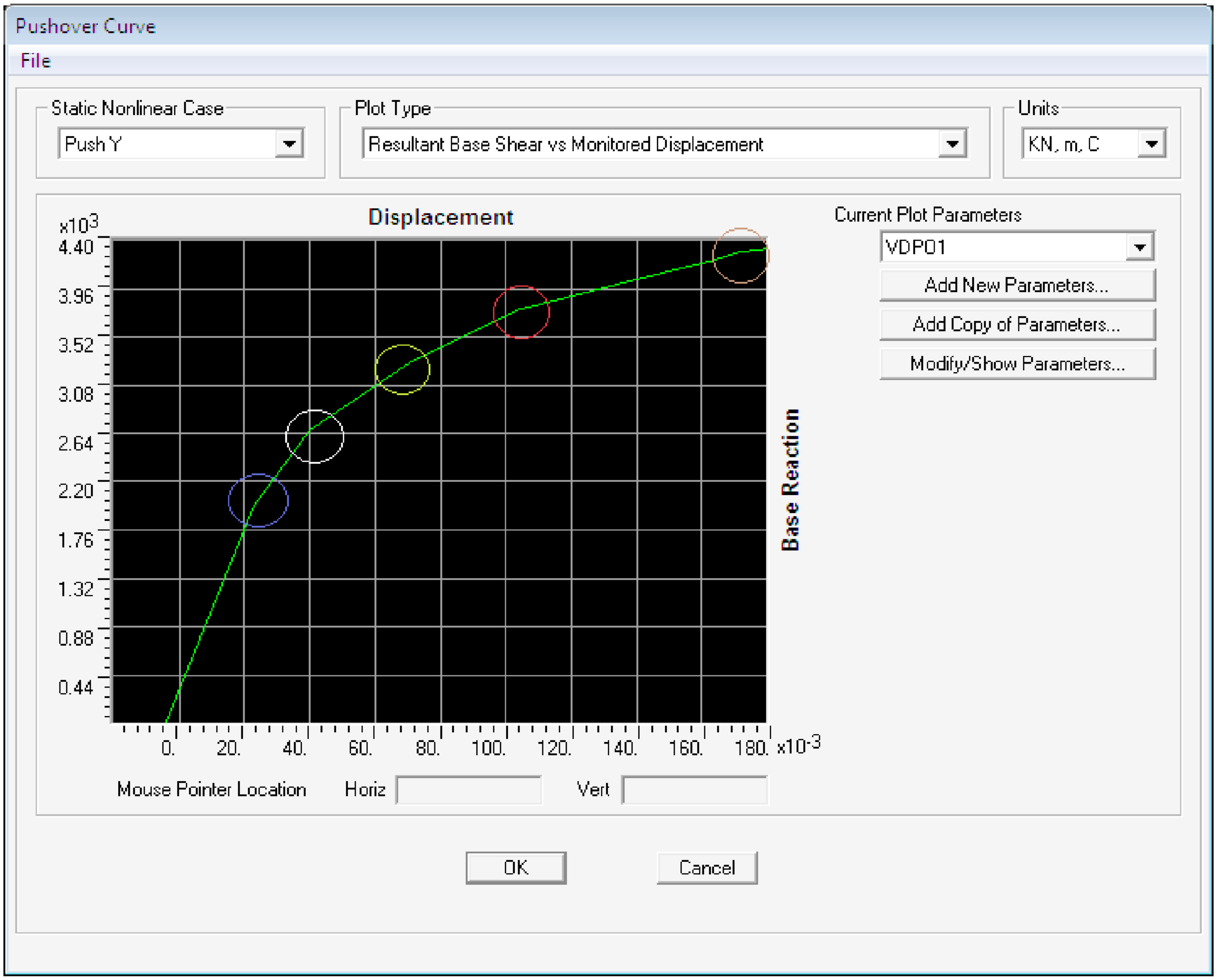

Figure 8-40- Base reaction-displacement Curve in Y direction

Again the points in which we can see a significant change in the slope of the graph are indicated by some circles around them. By comparing this capacity curve with the different performance levels specified in chapter 3 (figure 3-6) it becomes evident that this particular structure is within the “Immediate occupancy” performance range in y direction too.

B) ATC-40 capacity spectrum:

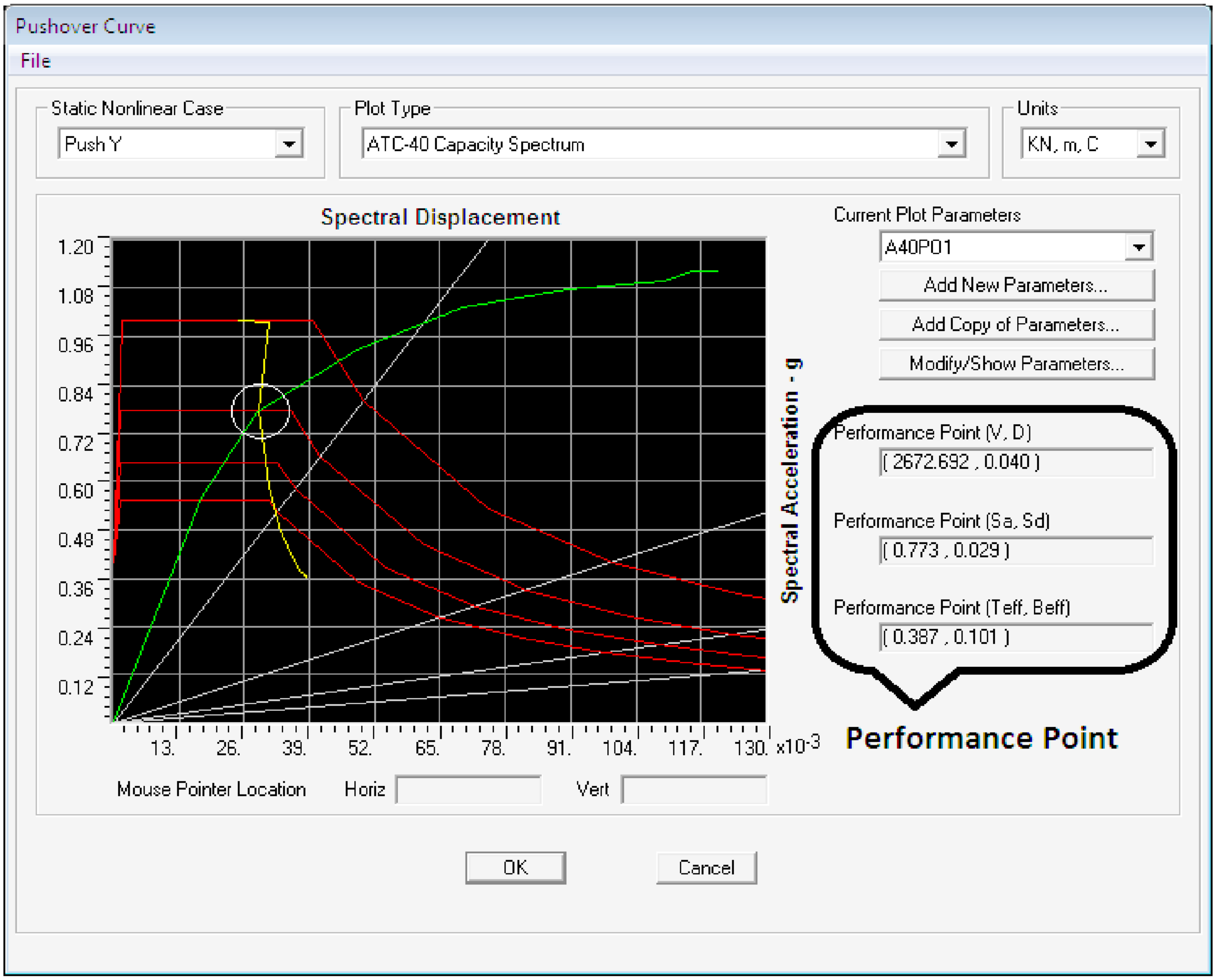

Figure 8-41- ATC-40 capacity spectrum Curve in Y direction

The point of intersection of the demand curve and the capacity curve that is the performance point is show with a circle around it. As it is shown in the right hand side of the graph, the coordinate of this point is: $S_a = 0.773, S_d = 0.029, or\ T_{eff} = 0.387\ , B_{eff} = 0.101$

8-2-2-3) Tabular Data:

A) Pushover curve tabular data:

TABLE: Pushover Curve - Push Y											
Step	Displacement	BaseForce	AtoB	BtoIO	IOtoLS	LStoCP	CPtoC	CtoD	DtoE	BeyondE	Total
	m	KN									
0	-0.003881	0	400	0	0	0	0	0	0	0	400
1	-0.003803	7.499	399	1	0	0	0	0	0	0	400
2	0.022727	1978.956	333	67	0	0	0	0	0	0	400
3	0.039821	2665.819	301	97	2	0	0	0	0	0	400
4	0.070659	3288.204	283	80	37	0	0	0	0	0	400
5	0.103233	3757.204	261	60	79	0	0	0	0	0	400
6	0.13386	4003.844	254	48	98	0	0	0	0	0	400
7	0.16407	4221.734	250	43	101	6	0	0	0	0	400
8	0.171606	4288.75	249	44	101	5	0	1	0	0	400
9	0.173825	4297.309	249	44	101	5	0	0	1	0	400
10	0.1757	4309.645	249	44	101	4	0	1	1	0	400
11	0.177692	4316.397	249	43	102	4	0	0	2	0	400
12	0.180445	4334.481	248	43	103	3	0	1	2	0	400

Table 8-3- Pushover curve tabular data in Y direction

B) Pushover curve demand capacity-ATC 40 tabular data:

TABLE: Pushover Curve Demand Capacity - ATC40 - Push Y								
Step	Teff	Beff	SdCapacity	SaCapacity	SdDemand	SaDemand	Alpha	PFPhi
			m		m			
0	0.315791	0.05	0	0	0.024772	1	1	1
1	0.315791	0.05	0.000051	0.002066	0.024772	1	0.719553	1.537407
2	0.353994	0.050322	0.017422	0.559686	0.030999	0.995859	0.700942	1.527286
3	0.386273	0.099755	0.028579	0.771069	0.028775	0.776372	0.685374	1.529194
4	0.458914	0.177792	0.048523	0.927528	0.030918	0.591009	0.702784	1.536166
5	0.521373	0.216508	0.069648	1.031449	0.032943	0.487865	0.722116	1.537953
6	0.580939	0.241504	0.090267	1.076731	0.035139	0.419148	0.737157	1.525927
7	0.633409	0.257399	0.109615	1.099866	0.037316	0.374425	0.760923	1.532195
8	0.642326	0.254988	0.115213	1.124167	0.037991	0.370684	0.756293	1.523154
9	0.646236	0.256515	0.116536	1.123355	0.038127	0.367522	0.75835	1.524899
10	0.649286	0.257443	0.117681	1.123763	0.038249	0.365243	0.760251	1.525993
11	0.652834	0.25881	0.118869	1.122801	0.038372	0.362452	0.762094	1.527508
12	0.657256	0.260042	0.120562	1.12352	0.038555	0.359295	0.764797	1.528889

Table 8-4- Pushover curve demand capacity-ATC 40 tabular data in Y direction

(9- SAP2000 ultimate software, version 15.0.0)

8-3) Assessment of the performance of the structure:

Based on the output information of the pushover analysis, the structure can be assessed to indicate at what level it can resist the seismic threat specified by the demand curve. Moreover, and if needed, one can decide a strengthening procedure to enhance the seismic performance of the structure up to level and cost agreed also with the owner on the bases of the expected damage for the primary and secondary structural members.

8-3-1) X direction:

From the previous section, it is clear that the coordinate of the performance point at the x direction is:

$$T_{eff} = 0.416\,, B_{eff} = 0.114$$

Referring to the tabular data, it can be seen that this point is located between the 3rd and the 4th step, and referring to the graphical representation of the hinges in different steps, it is clear that few hinges at the 4th step are in the “life safety” while most of the hinges are on safer levels around the Immediate Occupancy region:

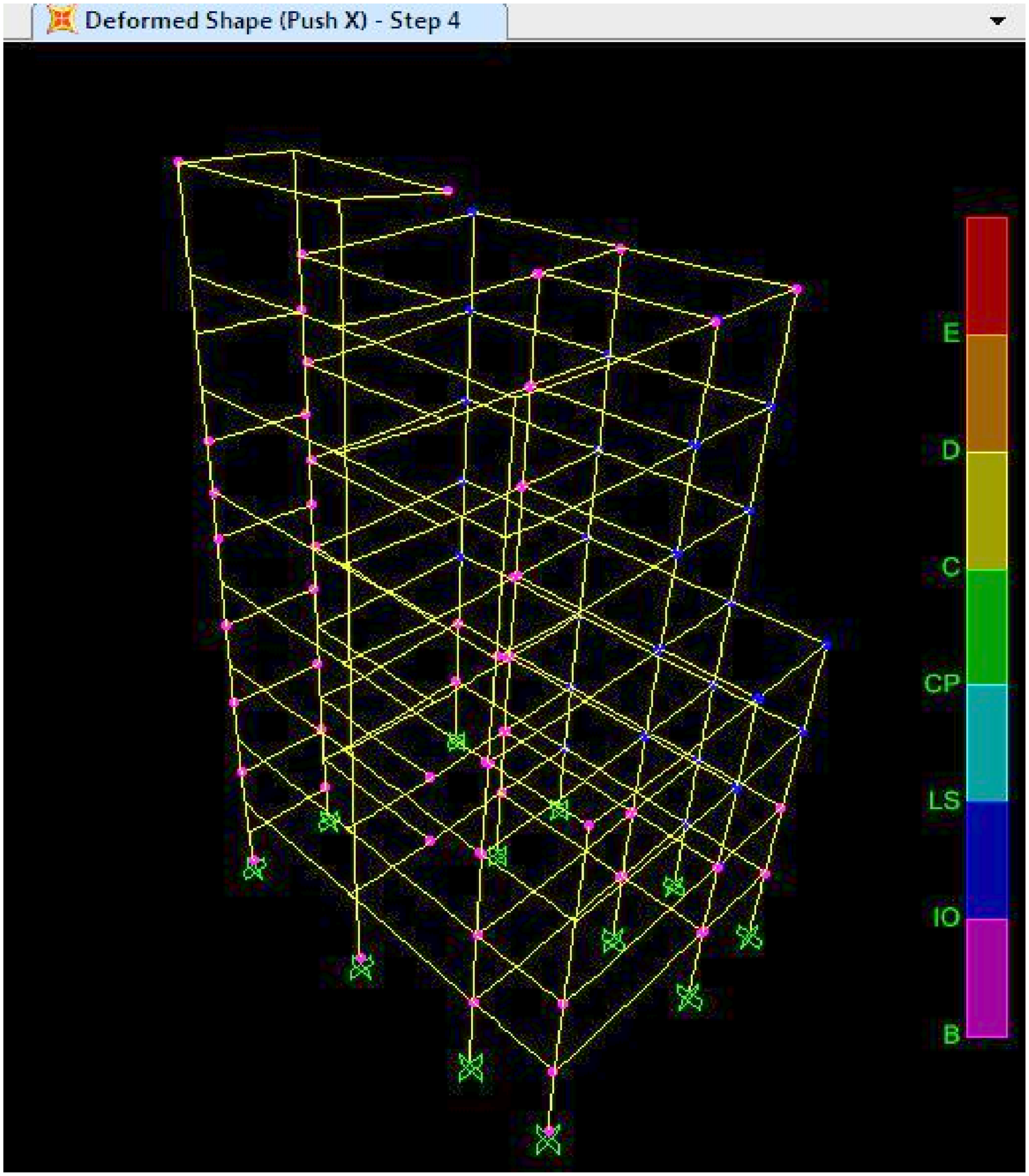

Figure 8-42- Created Hinges in the Structure at the Performance Point (X direction)

It is observed that most of the yielding and more intense inelastic deformations appear in the beams rather than the columns but unfortunately there are three columns at the base that rather prematurely are yielding. Although the hinges in these columns are before the "Immediate Occupancy" level and there is not any danger for the structure, for upgrading of the safety level, some kind of strengthening can be foreseeing towards a more uniform response at the lower floors.

8-3-2) Y direction:

From the previous section, it is clear that the coordinate of the performance point at the y direction is:

$$T_{eff} = 0.387 \, , B_{eff} = 0.101$$

Referring to the tabular data, it can be seen that this point is located between the 3rd and the 4th steps, and referring to the graphical representation of the hinges in different steps, it is clear that all of the hinges at the 4th step are in the "life safety" or safer levels:

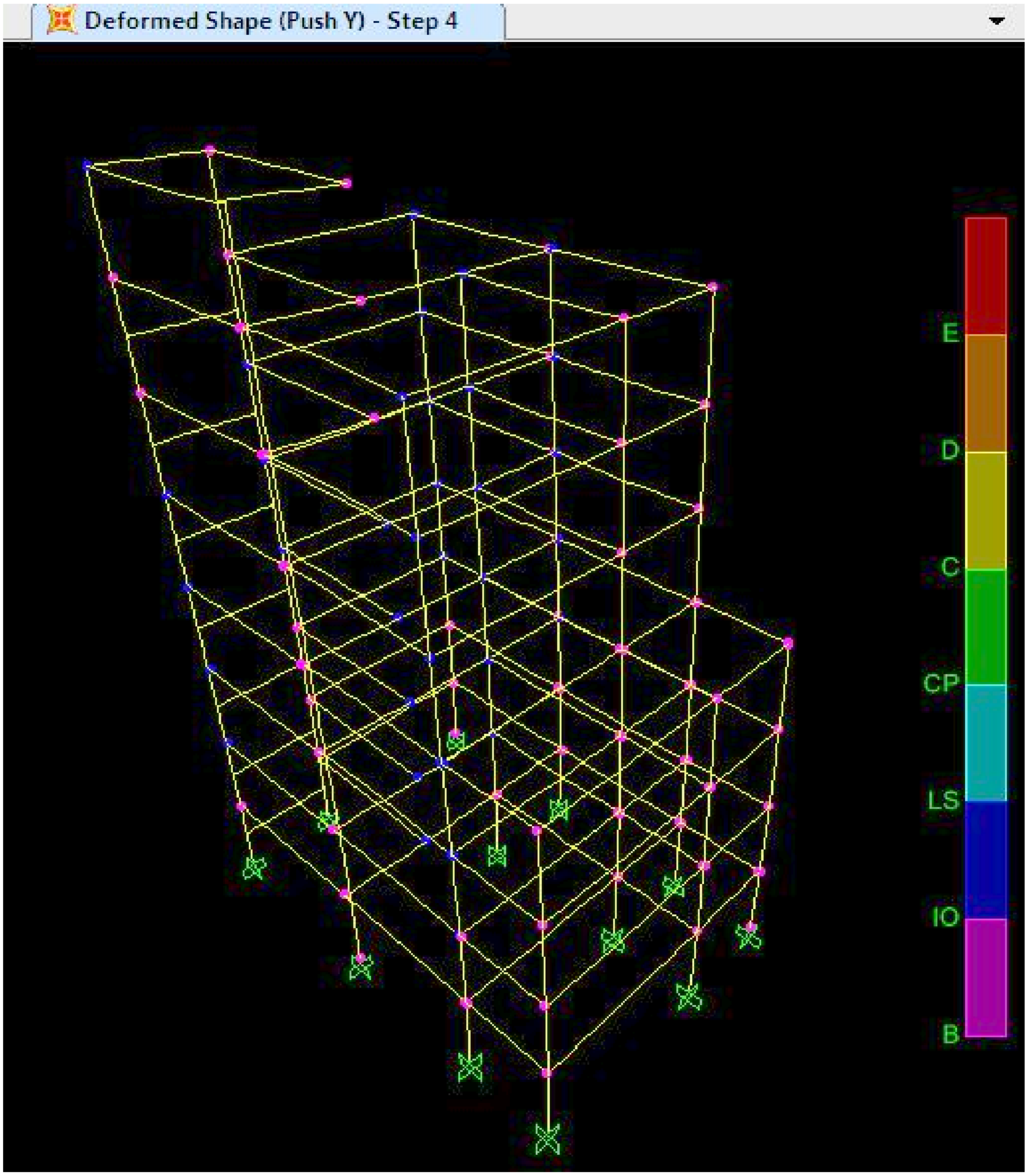

Figure 8-43- Created Hinges in the Structure at the Performance Point (Y direction)

Again most of the yielding is manifested in the beams rather than the columns but unfortunately there are three sections at the columns of the 1st floor (at the base) and as it is mentioned before are yielding rather prematurely which is not favorable. Again it can be mentioned that although yielding in these columns are before the "Immediate Occupancy" level and there is not any real threat for the structure, for upgrading of the safety level, some kind of strengthening can be applied to the structure. **(9- SAP2000 ultimate software, version 15.0.0)**

9) Strengthening of the structure:

The overall performance of the structure results quite satisfactory and according to the code characterization it can be categorized in the “Immediate Occupancy” category. Therefore according to the codes the building safely can sustain the anticipated earthquake loads with minor damage.

From the analysis though some particular features of this building were revealed that destruct the uniform behavior of its elements favoring the formation of plastic hinges in certain areas. This, from the owner side, may require some minor interventions to improve further the earthquake response of the building to withstand the same level of seismic demands with less damage, or higher levels of demand.

9-1) Strategy Selection:

Because the problem is initiating from the base columns, increasing of the cross section areas by jacketing can be selected as one strengthening strategy. A one-sided concrete layer with a new row of rebars is added as below to all of these column’s cross sections:

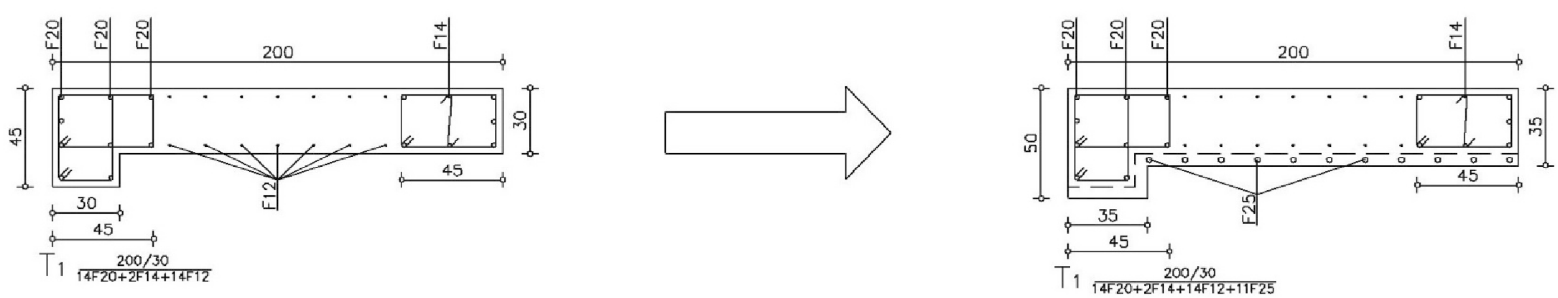

Figure 9-1- T1 Column Cross Section Area-Before and After the Strengthening

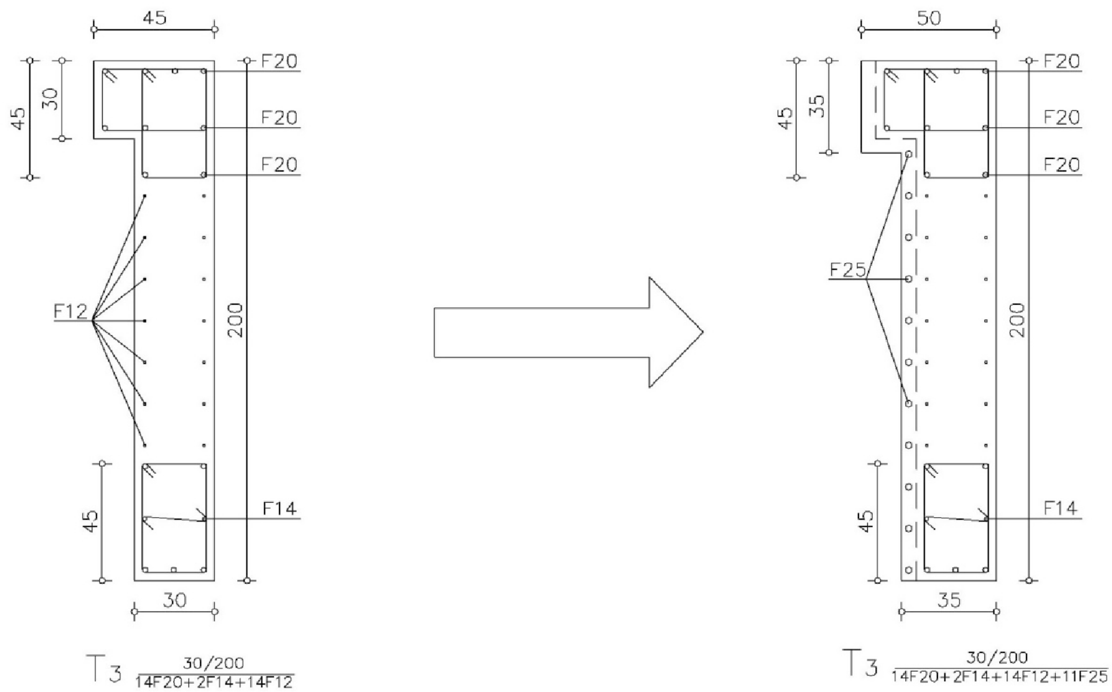

Figure 9-2- T3 Column Cross Section Area-Before and After one sided strengthening

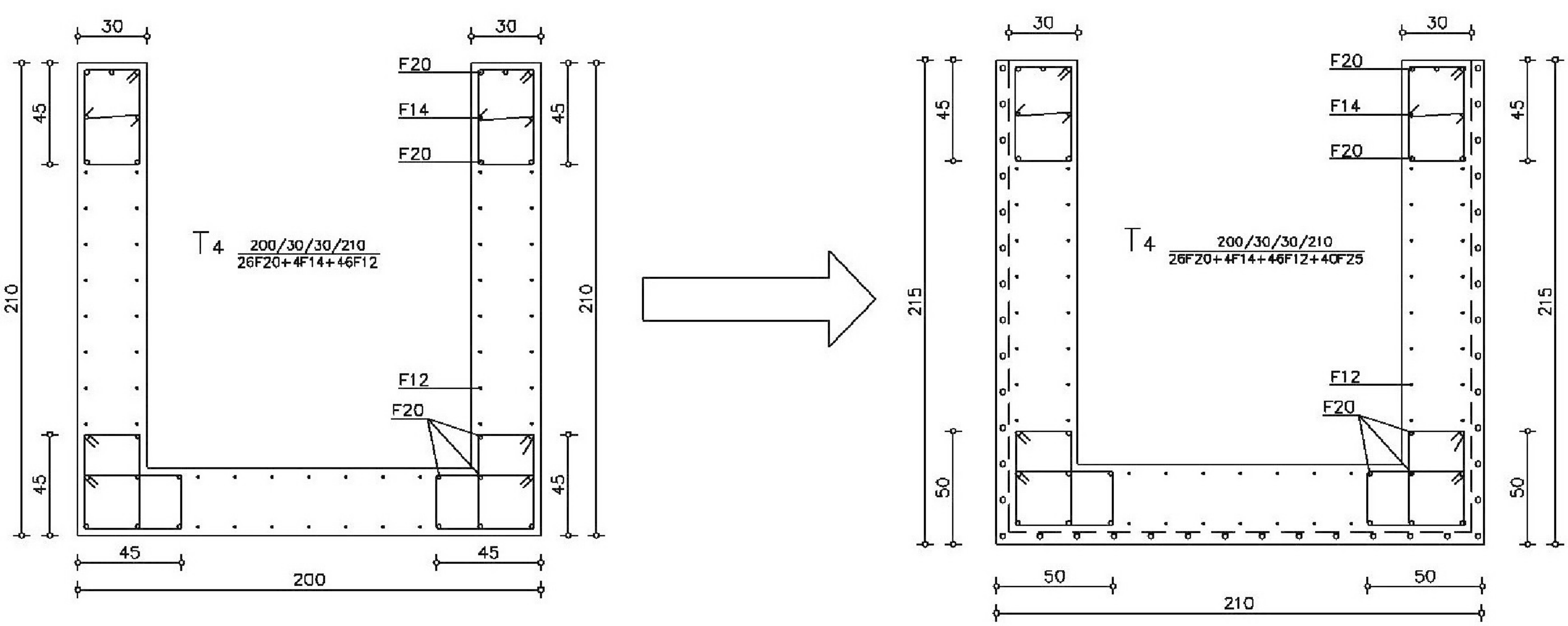

Figure 9-3- T4 Column Cross Section Area-Before and After the Strengthening

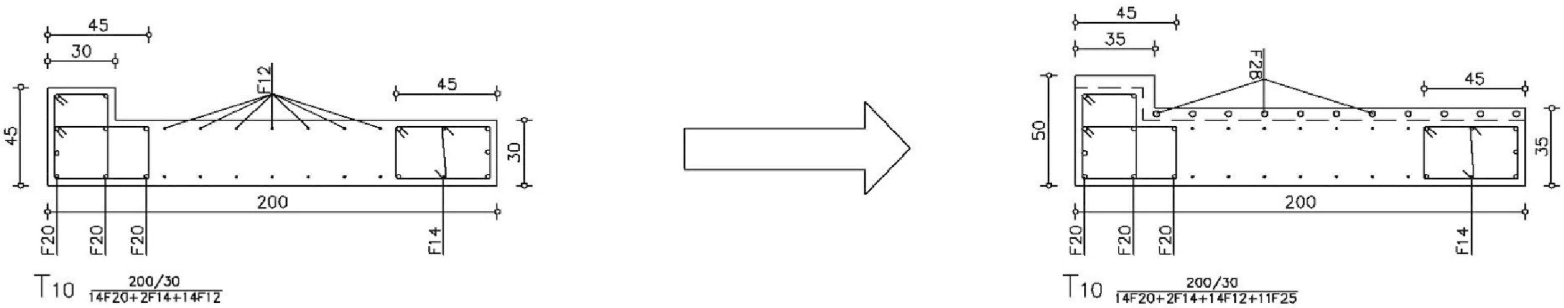

Figure 9-4- T10 Column Cross Section Area-Before and After the Strengthening

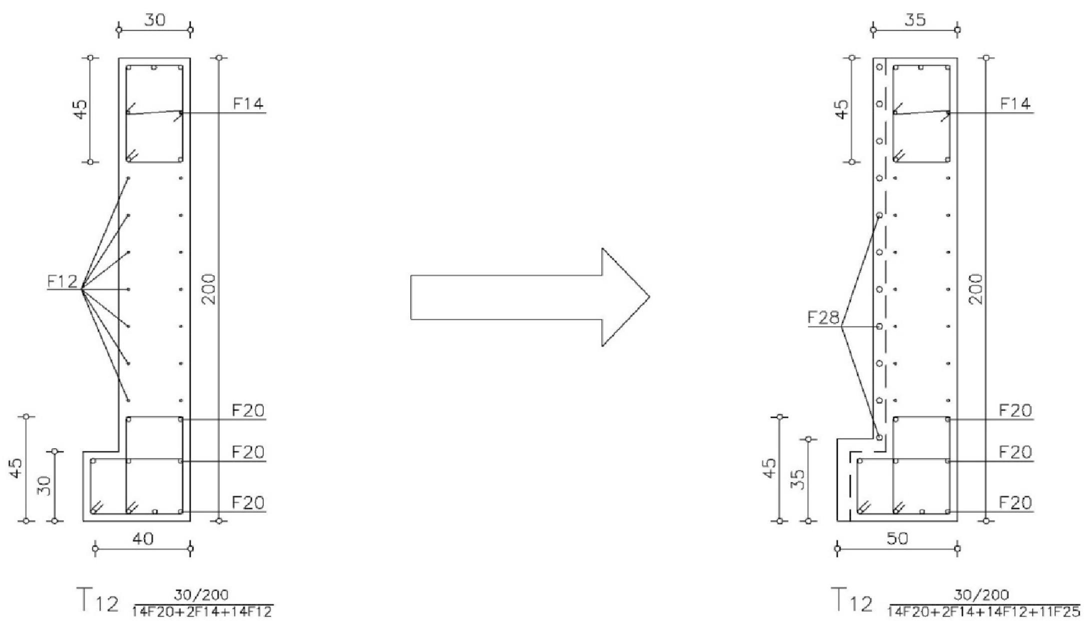

Figure 9-5- T12 Column Cross Section Area-Before and After the Strengthening

9-2) Re-analysis outputs:

After re-analysis of the structure, the outputs are as below:

9-2-1) X direction:

9-2-1-1) Pushover Curves:

A) Base reaction-displacement:

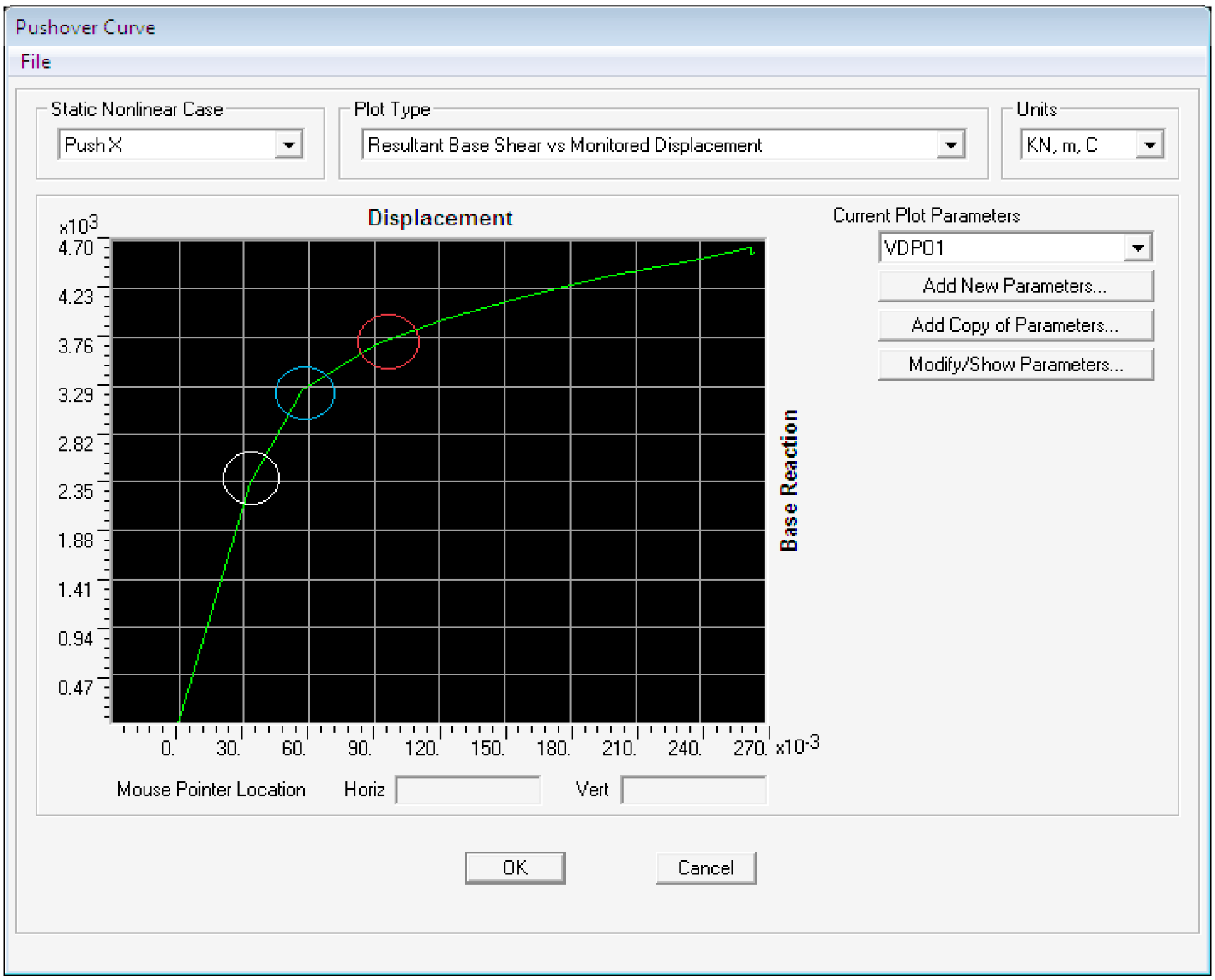

Figure 9-6- Base reaction-displacement Curve in X direction

Again the points in which we can see a significant change in the slope of the graph are indicated by some circles around them. By comparing this capacity curve with the different performance levels specified in chapter 3 (figure 3-6) it becomes evident that this particular structure is within the "Immediate occupancy" performance range in x direction.

B) ATC-40 capacity spectrum:

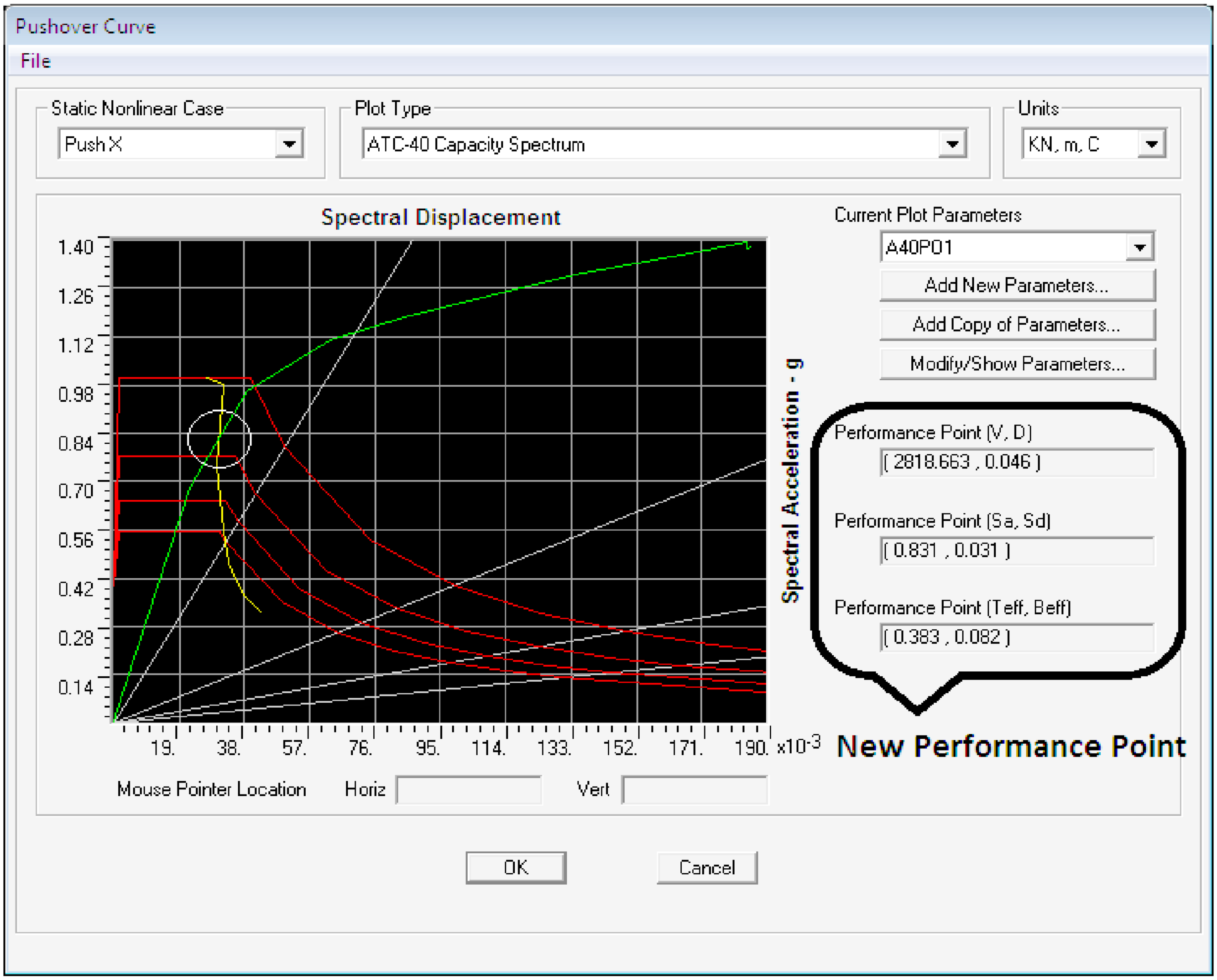

Figure 9-7- ATC-40 capacity spectrum Curve in X direction

The point of intersection of the demand curve and the capacity curve that is the performance point is shown with a circle around it. As it is shown in the right hand side of the graph, the coordinate of this point is: $S_a = 0.831, S_d = 0.031, or\ T_{eff} = 0.383\ , B_{eff} = 0.082$

9-2-1-2) Tabular Data:

A) Pushover curve tabular data:

TABLE: Pushover Curve - Push X											
Step	Displacement	BaseForce	AtoB	BtoIO	IOtoLS	LStoCP	CPtoC	CtoD	DtoE	BeyondE	Total
	m	KN									
0	-0.000217	0	400	0	0	0	0	0	0	0	400
1	0.000723	89.889	399	1	0	0	0	0	0	0	400
2	0.032777	2339.364	343	57	0	0	0	0	0	0	400
3	0.057159	3255.078	311	82	7	0	0	0	0	0	400
4	0.091093	3698.804	289	64	47	0	0	0	0	0	400
5	0.121578	3934.445	274	65	61	0	0	0	0	0	400
6	0.156343	4136.669	266	52	82	0	0	0	0	0	400
7	0.195955	4344.677	256	42	102	0	0	0	0	0	400
8	0.239604	4529.177	250	39	78	33	0	0	0	0	400
9	0.262898	4633.863	248	34	77	40	0	1	0	0	400
10	0.262901	4571.496	248	34	77	40	0	0	1	0	400
11	0.263063	4573.518	248	34	77	40	0	0	1	0	400
12	0.265299	4586.76	248	34	77	39	0	1	1	0	400

Table 9-1- Pushover curve tabular data in X direction

B) Pushover curve demand capacity-ATC 40 tabular data:

TABLE: Pushover Curve Demand Capacity - ATC40 - Push X								
Step	Teff	Beff	SdCapacity	SaCapacity	SdDemand	SaDemand	Alpha	PFPhi
			m		m			
0	0.32934	0.05	0	0	0.026943	1	1	1
1	0.32934	0.05	0.000693	0.025725	0.026943	1	0.685682	1.356169
2	0.361132	0.052708	0.02211	0.68249	0.031781	0.980999	0.672637	1.492227
3	0.402151	0.109483	0.038796	0.965708	0.029991	0.746526	0.661447	1.478903
4	0.477909	0.190237	0.062988	1.110208	0.031723	0.559137	0.653787	1.449648
5	0.537039	0.227998	0.084443	1.178671	0.033247	0.464063	0.655044	1.442325
6	0.594576	0.24552	0.108863	1.239662	0.035722	0.406778	0.654828	1.438139
7	0.649906	0.2546	0.136703	1.302915	0.038463	0.366593	0.654366	1.435022
8	0.703424	0.260499	0.167307	1.361187	0.041233	0.335465	0.652952	1.433421
9	0.728623	0.261416	0.183983	1.39512	0.042647	0.323384	0.651795	1.430104
11	0.733358	0.265867	0.18406	1.37746	0.042629	0.319022	0.651556	1.430445
12	0.733432	0.265777	0.185637	1.381562	0.042763	0.318251	0.651555	1.430403

Table 9-2- Pushover curve demand capacity-ATC 40 tabular data in X direction

9-2-2) Y direction:

9-2-2-1) Pushover Curves:

A) Base reaction-displacement:

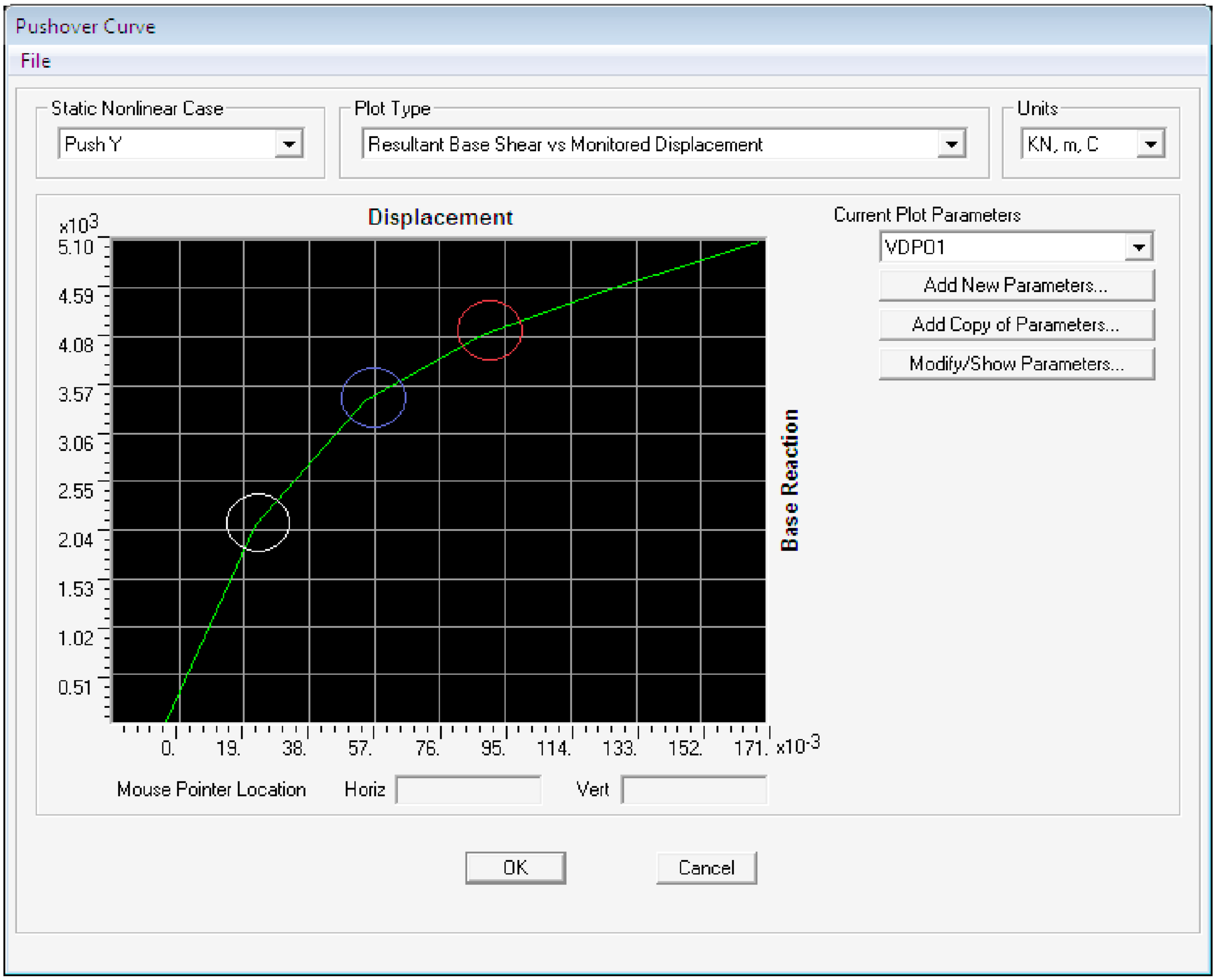

Figure 9-8- Base reaction-displacement Curve in Y direction

Again the points in which we can see a significant change in the slope of the graph are indicated by some circles around them. By comparing this capacity curve with the different performance levels specified in chapter 3 (figure 3-6) it becomes evident that this particular structure is within the “Immediate occupancy” performance range in y direction too.

B) ATC-40 capacity spectrum:

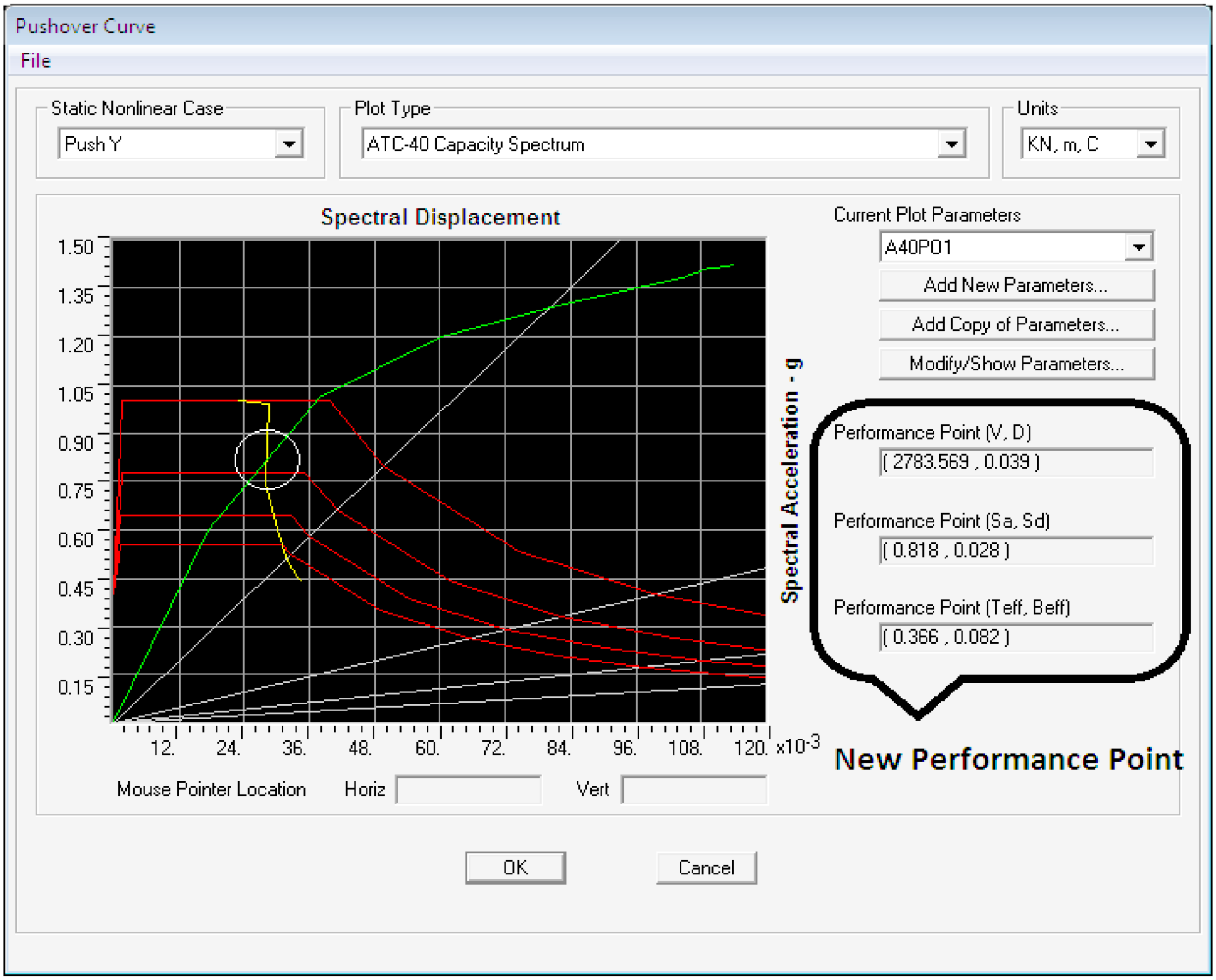

Figure 9-9- ATC-40 capacity spectrum Curve in Y direction

The point of intersection of the demand curve and the capacity curve that is the performance point is shown with a circle around it. As it is shown in the right hand side of the graph, the coordinate of this point is: $S_a = 0.818, S_d = 0.028, or\ T_{eff} = 0.366\ , B_{eff} = 0.082$

Finally to quantify the effect of the proposed strengthening numerically, the following calculations can be done. The following coordinates for the performance points can be extracted for both directions:

X direction:

$$Before\ strengthening : S_a = 0.719, S_d = 0.031$$

$$After\ strengthening : S_a = 0.831, S_d = 0.031$$

Percentage of increase:

$$\left(\frac{0.831-0.719}{0.719}\right)\times 100 = \%\ 15.57$$

Y direction:

$$Before\ strengthening : S_a = 0.773, S_d = 0.029$$

$$After\ strengthening : S_a = 0.818, S_d = 0.028$$

Percentage of increase:

$$\left(\frac{0.818-0.773}{0.773}\right)\times 100 = \%\ 5.82$$

This means that the proposed strengthening, in these specific elements, increased the capacity of the structure by almost 15.57% in spectral acceleration for practically the same displacement for X direction and 5.82% in spectral acceleration for practically the same displacement in Y direction.

9-2-2-2) Tabular Data:

A) Pushover curve tabular data:

TABLE: Pushover Curve - Push Y											
Step	Displacement	BaseForce	AtoB	BtoIO	IOtoLS	LStoCP	CPtoC	CtoD	DtoE	BeyondE	Total
	m	KN									
0	-0.003901	0	400	0	0	0	0	0	0	0	400
1	-0.003671	23.279	399	1	0	0	0	0	0	0	400
2	0.022378	2087.993	335	65	0	0	0	0	0	0	400
3	0.053972	3397.388	297	86	17	0	0	0	0	0	400
4	0.088169	4091.584	277	61	62	0	0	0	0	0	400
5	0.122341	4551.252	267	42	91	0	0	0	0	0	400
6	0.15515	4932.373	259	43	93	5	0	0	0	0	400
7	0.162465	5018.689	255	46	93	5	0	1	0	0	400
8	0.164065	5029.028	254	46	94	5	0	0	1	0	400
9	0.169245	5084.151	254	46	94	2	0	3	1	0	400

Table 9-3- Pushover curve tabular data in Y direction

B) Pushover curve demand capacity-ATC 40 tabular data:

TABLE: Pushover Curve Demand Capacity - ATC40 - Push Y								
Step	Teff	Beff	SdCapacity	SaCapacity	SdDemand	SaDemand	Alpha	PFPhi
			m		m			
0	0.304018	0.05	0	0	0.022959	1	1	1
1	0.304018	0.05	0.00015	0.006515	0.022959	1	0.701219	1.537333
2	0.339873	0.050926	0.017221	0.600143	0.028466	0.992031	0.682737	1.526003
3	0.388408	0.109614	0.037885	1.010939	0.027961	0.746142	0.659477	1.527612
4	0.448817	0.171673	0.060158	1.202245	0.030135	0.602242	0.667848	1.530466
5	0.506014	0.215968	0.082714	1.300443	0.032003	0.503163	0.686782	1.526247
6	0.55103	0.230812	0.104434	1.384612	0.033946	0.450067	0.699048	1.522989
7	0.559021	0.230583	0.109603	1.411901	0.034452	0.443811	0.697534	1.517899
8	0.561295	0.231657	0.110605	1.413286	0.034528	0.441189	0.698286	1.518611
9	0.567641	0.233372	0.113961	1.423803	0.034815	0.434966	0.700725	1.519333

Table 9-4- Pushover curve demand capacity-ATC 40 tabular data in Y direction

(9- SAP2000 ultimate software, version 15.0.0)

9-3) Re-assessment of the structure:

Now based on the output information from the re-analysis, the structure can be re-assessed to see that if the enhancement strategy is effective or not.

9-3-1) X direction:

From the previous section, it is clear that the coordinate of the new performance point at the x direction is:

$$T_{eff} = 0.383\ , B_{eff} = 0.082$$

Referring to the new tabular data for the x direction, it can be seen that this point is located between the 2nd and the 3rd steps, and referring to the graphical representation of the hinges in different steps, it is clear that all of the hinges at the 3rd step are in the "life safety" or safer levels:

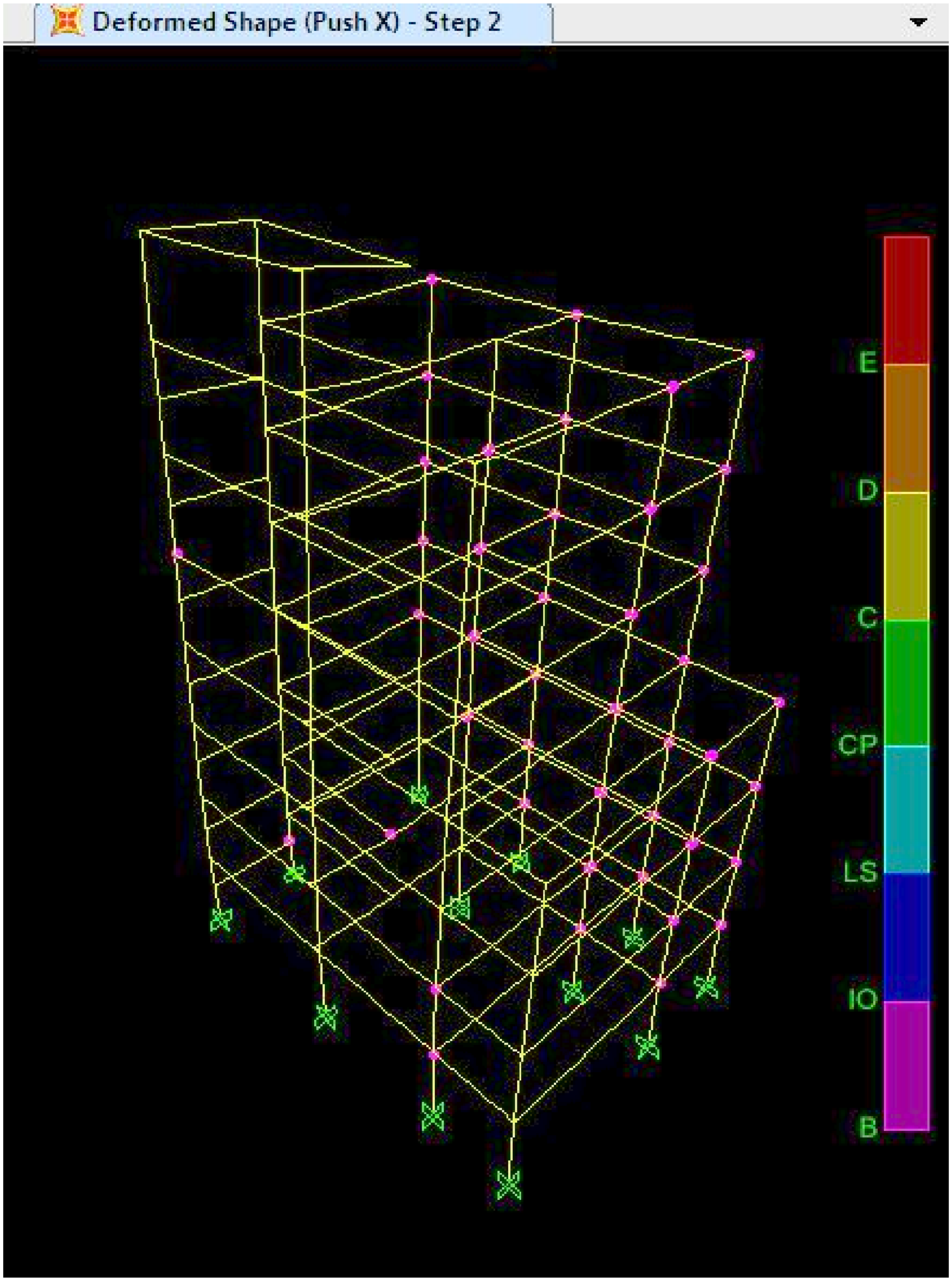

Figure 9-10- Created Hinges in the Structure around the Performance Point (X direction-step 2)

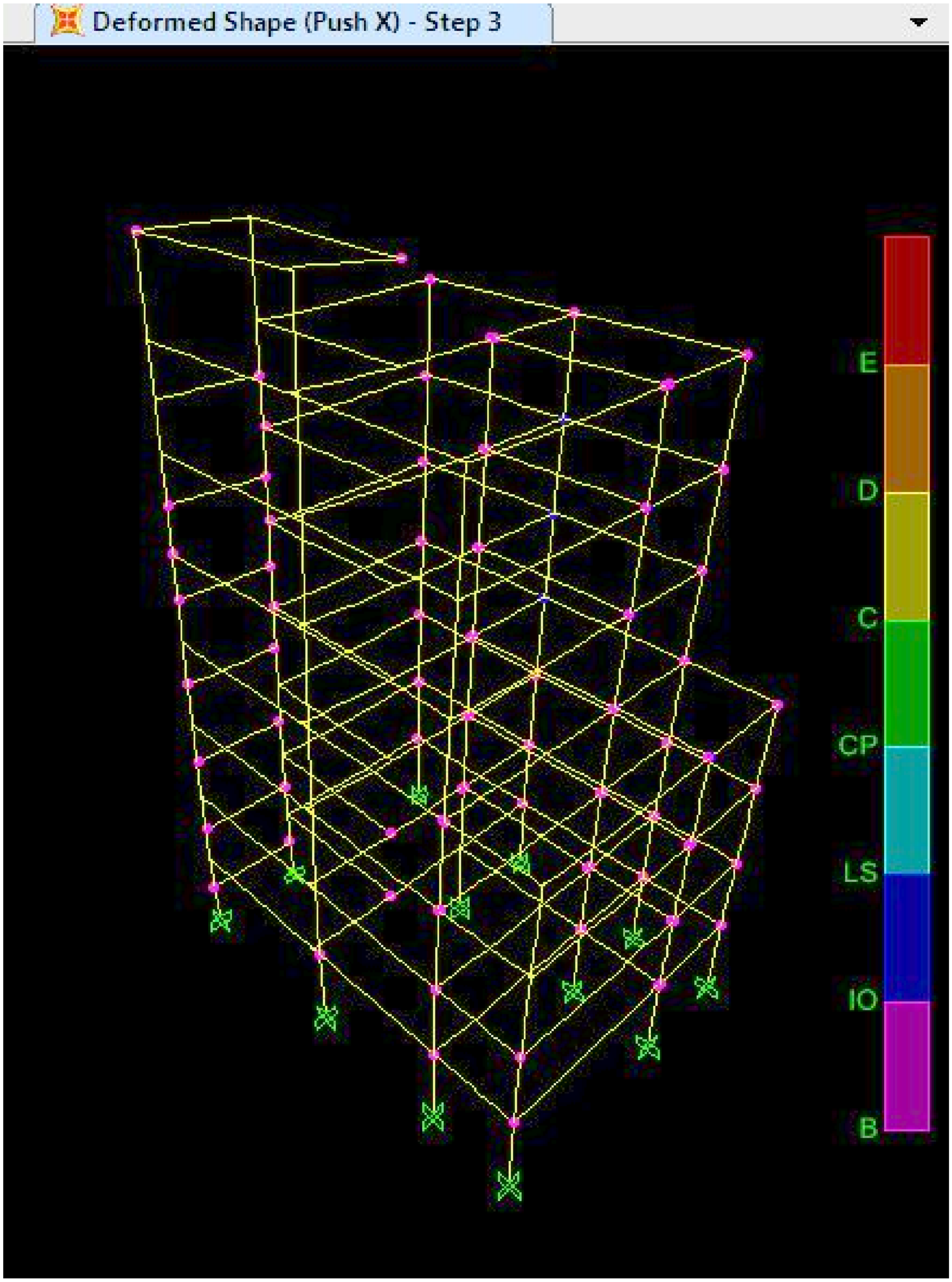

Figure 9-11- Created Hinges in the Structure around the Performance Point (X direction-step 3)

Fortunately all of the hinges are created in the beams and the three hinges that were created at the columns of the basement (at the base) in the original structure, have disappeared. So strengthening is effective in this direction.

9-3-2) Y direction:

From the previous section, it is clear that the coordinate of the new performance point at the y direction is:

$$T_{eff} = 0.366\,, B_{eff} = 0.082$$

Referring to the new tabular data for the y direction, it can be seen that this point is located between the 2nd and the 3rd steps, and referring to the graphical representation of the hinges in different steps, it is clear that all of the hinges at the 3rd step are in the "life safety" or safer levels:

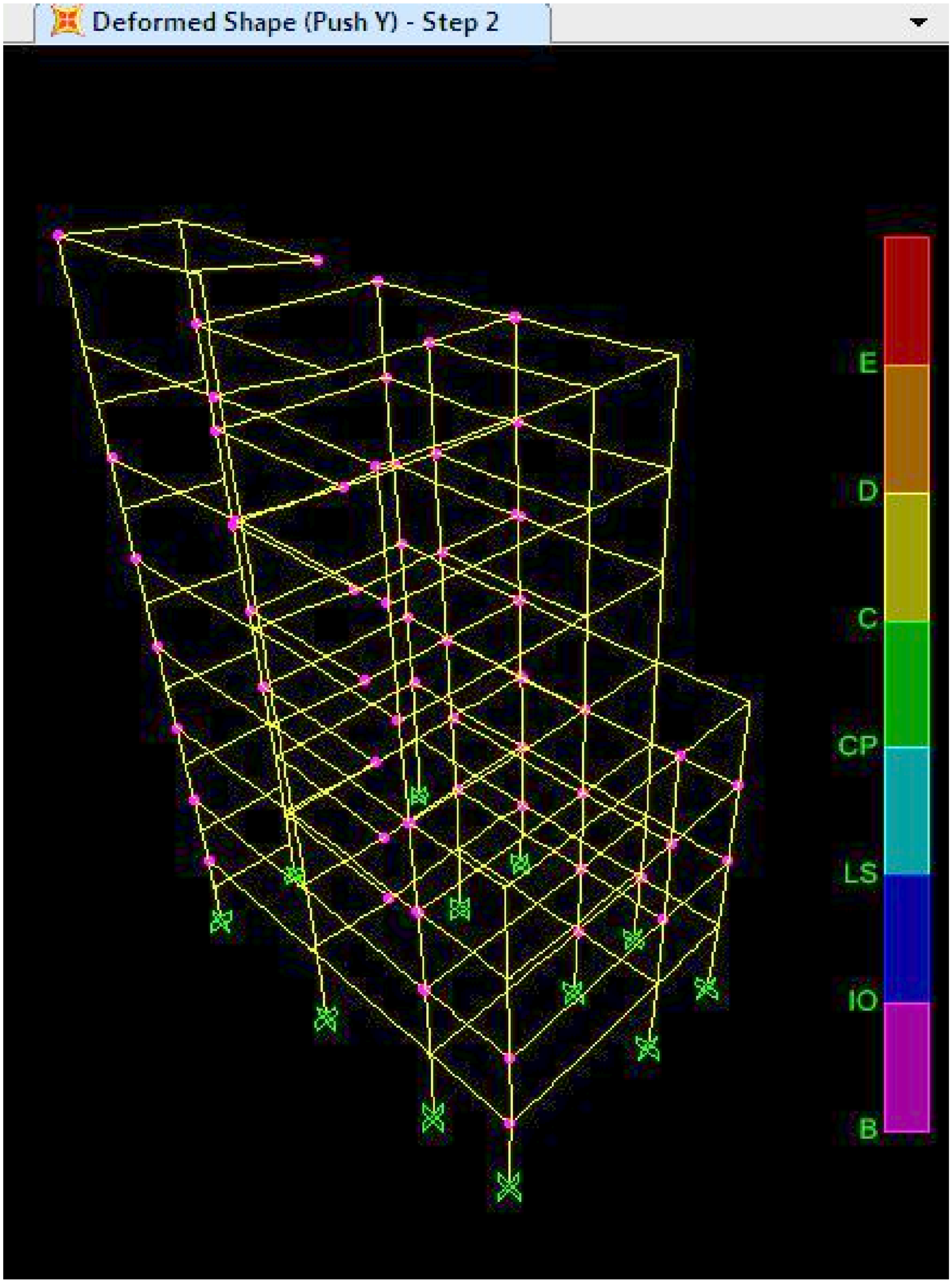

Figure 9-12- Created Hinges in the Structure around the Performance Point (Y direction-step 2)

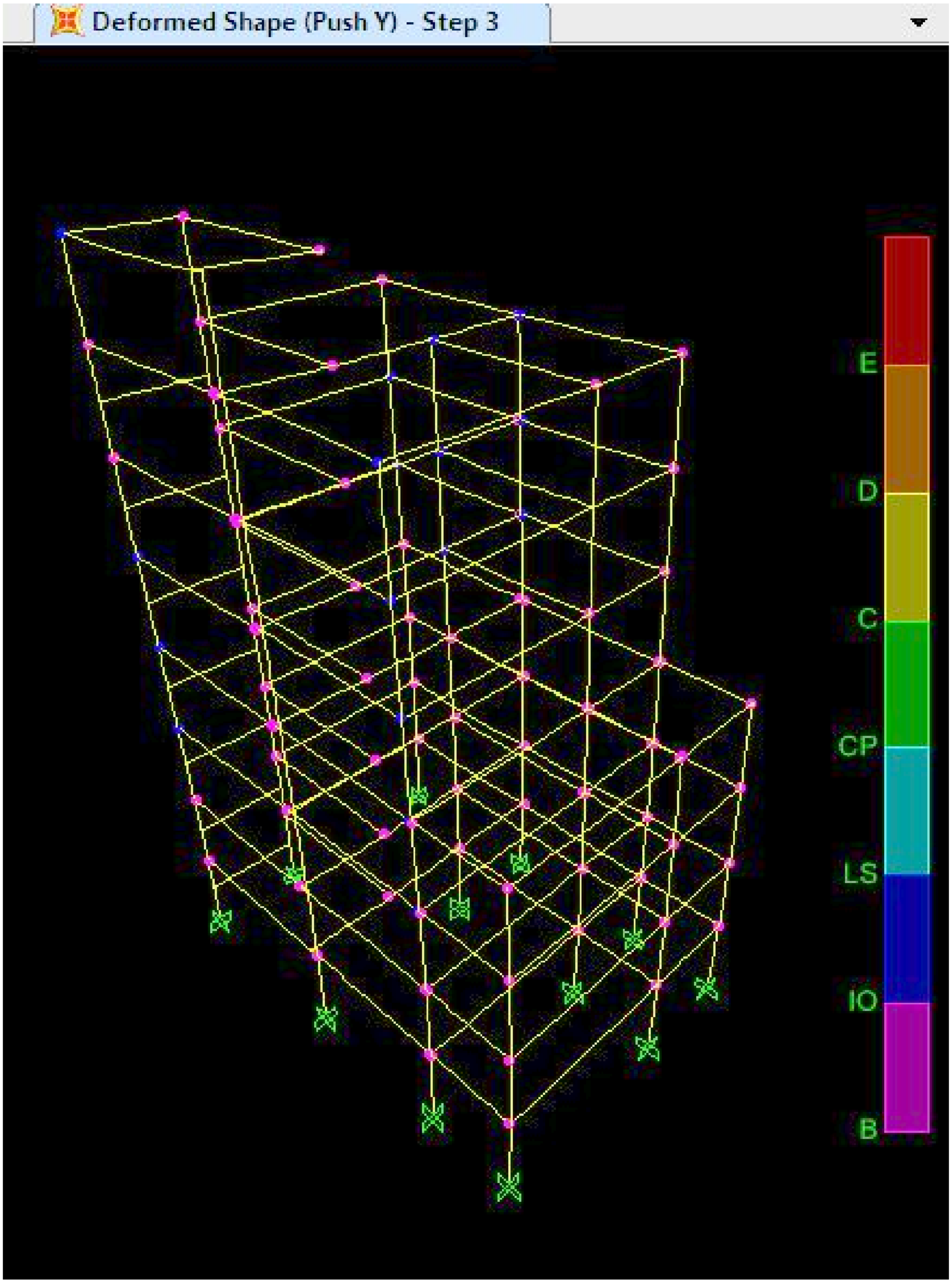

Figure 9-13- Created Hinges in the Structure around the Performance Point (Y direction-step 3)

Fortunately all of the hinges are now created in the beams and the three hinges that were created at the columns of the basement (at the base) in the original structure, have disappeared. So strengthening is effective in this direction too. **(9- SAP2000 ultimate software, version 15.0.0)**

This is indicative of the role of the designer who has to intervene in a targeted way with cost effective but also feasible methods and quantify the overall performance of the initial and the strengthened structure.

10) Conclusions

In this work the main features of spectral analysis and push over analysis were discussed for the linear and non linear static analysis of buildings that constitute the primal methods for the design of new and evaluating the performance of existing buildings respectively. Based on these methods the analysis of a specific building was performed using Sap 2000 software.

The structure, at its performance point in both directions, is in a good and acceptable condition because at that point the plastic hinges formed are not at their critical state. The overall status of the building can be categorized as “Immediate Occupancy” stage and as it was mentioned before, no serious rehabilitation process is required. This is because this structure is a new one and has been designed on the basis of the new codes.

In the following, some of the most important features of the structure are mentioned and an overall assessment will be done to clarify the deficiencies of the structure.

In the x direction the structure has two bays with a total length of 9.8m and in the y direction three bays with a total length of 14.05m. The structure is made of complete frames in the y direction while this is not the case in the x direction. The first frame is shorter while the other frames are taller. Also the stiffness of the structure in the x direction is lower than the stiffness in the y direction.

Based on the above characteristics, some of the most critical areas in the structure are identified as follows:

1. From the figures presented in the previous chapters it becomes evident that the process of creation of the plastic hinges started from right hand side of the structure and as the loading increases, the propagation of the hinges is extended from the right hand side of the structure to the left hand side. Finally in the last steps, the severity of damage is more intense and the hinges in that part reach the more critical levels such as “collapse” level. Therefore, it seems that an event of partial collapse in this side of the structure is more likely to happen or in other words a program of rehabilitation in this side of the structure has the first priority.

2. Another weak area that is common in the most of the structures is the beams of the stairs (These beams are located at the mid height of the stories). Again it is clear that the hinges in this area initiated sooner than the other parts of the structure, so this part of the structure needs more attention in a probable future rehabilitation plan.

3. The base of the columns at the basement (The connection points of the columns to the foundation) should be critical and require some additional consideration. It is obvious that these points are more vulnerable than the other points in the columns in the whole structure because these are the only points that some hinges are unavoidably initiated in the columns. So more attention is needed in these areas to allow for the formation of plastic hinges but avoid collapse and anchorage failure by losing the bond between the rebars and the concrete.

4. Also it is observed that the hinges in the shorter frames that are located in front of the structure, are more than the other part and they reach more critical levels. This is due to the non canonical form of the structure in its plan.

5. Finally, strengthening of particular elements at lower levels can increase substantially the carrying capacity of the structure establishing a more unified behavior and distribution of damage.

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