BEHAVIOUR OF R.C.C FRAMES DESIGNED FOR DIFFERENT LEVELS OF DUCTILITY

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THESIS CERTIFICATE

This is to certify that the thesis entitled "**BEHAVIOUR OF R.C.C FRAMES DESIGNED FOR DIFFERENT LEVELS OF DUCTILITY**" submitted by **AKANSHA BINDAL** bearing enrolment no. **132658** to Jaypee University of Information Technology, Waknaghat for the award of the degree of Master of Technology is a bonafide record of research work carried out by her under my supervision. The contents of this thesis, in full or in parts, have not been submitted to any other Institute or University for the award of any degree or diploma.

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ABSTRACT

Keywords: Pushover analysis, Target displacement, yield displacement, ductility

In this thesis four, two dimensional R.C.C frames are taken with different material properties The performance of these frames are compared using Pushover analysis using ETABS 2015 software The results in terms of storey displacement, ductility, drift, sequence of cracking and yielding and damage potential are studied. The ductility for all frames are calculated using target displacement and yield displacement values obtained from ASCE 41-13 curves from Pushover analysis. It is concluded that the frame with M40 grade concrete and Fe 415 steel is performing best and is therefore maximum ductile as compared to other frames.

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ABBREVIATIONS

ACI - American Concrete Institute

ATC - Applied Technology Council

BS - British Standard

CQC - Complete Quadratic Combination

CSM - Capacity Spectrum Method

DCM - Displacement Coefficient Method

EC - Eurocode

FEMA - Federal Emergency Management Agency

IS - Indian Standard

MDOF - Multi Degree of Freedom

PA- Pushover analysis

MODE 1 - Fundamental Mode Shape as Load Pattern

MPA - Modal Pushover Analysis

PGA - Peak Ground Acceleration

RC - Reinforced Concrete

ETABS- Extended Three Dimensional Building Systems

SDOF - Single Degree of Freedom

SRSS - Square Root of Sum of the Square.

NOTATION

SYMBOLS

a - regression constant
c - classical damping
C0 - factor for MDOF displacement
C1 - factor for inelastic displacement
C2 - factor for strength and stiffness degradation
C3 - factor for geometric nonlinearity
d - effective depth of the section
db - diameter of the longitudinal bar
dp - spectral displacement corresponding to performance point
D - overall depth of the beam.
D (*t*) *n* - displacement response for an equivalent SDOF system,
Ec - short-term modulus of elasticity of concrete
ED - energy dissipated by damping
Es - modulus of elasticity of steel rebar
ES - maximum strain energy
E sec - elastic secant modulus
EI - flexural rigidity of beam
fc - concrete compressive stress
f ‘cc- compressive strength of confined concrete
f ‘co- unconfined compressive strength of concrete
fck - characteristic compressive strength of concrete
Fe - elastic strength
fy - yield stress of steel rebar
Fy - defines the yield strength capacity of the SDOF
F1- frame 1 considered for study
F2- Frame 2 considered for study
F3- Frame 3 considered for study
F4 – Frame4 considered for study
G - shear modulus of the reinforced concrete section
h - overall building height (in m)
k - lateral stiffness
lp - equivalent length of plastic hinge
m - storey mass
N - number of modes considered
Peff (*t*) - effective earthquake force
Sa - spectral acceleration

Sd - spectral displacement
T - fundamental natural period of vibration

GREEK SYMBOLS

α - post-yield stiffness ratio

βeq - equivalent damping

βi - initial elastic damping

βs - damping due to structural yielding

δt- target displacement

$\in sm$ - steel strain at maximum tensile stress

$\{\emptyset n\}$ - nth mode shape of the structure

ϕu - ultimate curvature

ϕy - yield curvature

μ - displacement ductility ratio

θ_p - plastic rotation

θ_u- ultimate rotation

θ_y - yield rotation

CHAPTER 1
INTRODUCTION

1.1 GENERAL

In India almost 55% of the country is prone to earthquakes mild, moderate and catastrophic. Due to these earthquakes many buildings suffer damage and even collapse without warning leading to casualities.So taking all these things into account the buildings should be designed according to earthquake resistant design philosophy. This includes fundamental basis of design, design loads, choice of materials and analysis techniques. Since the earthquake forces are dynamic in nature earthquake displacements should be taken into account. Some objectives of philosophy are-

1. To resist minor earthquake without any damage.
2. To resist moderate earthquake without structural damage but with some non-structural damage.
3. To resist major or severe earthquake without major failure or collapse.

Limit States of Earthquake Resistant Design-

1. <u>Serviceability Limit State-</u> This limit aims to ensure adequate strengths in all components of the structure to resist earthquake induced forces while remaining elastic.
2. <u>Damage Control Limit State-</u> This allows economically repairable damage and life threatening damages should not occur.
3. <u>Survival Limit State-</u> According to this life threatening collapse of structure should be prevented in case of severe earthquakes.

1.2 SEISMIC RESPONSE OF BUILDINGS

Earthquake motion is vibratory and cyclic about equilibrium position and is dynamic in nature. It is time dependent. The building shakes in horizontal and vertical directions, it rocks, twists and distorts. The structural response depends on nature of excitation and dynamic characteristics of the building. Natural frequency/ natural periods mainly determine the response of structure. Tall buildings have higher time period than low or medium rise buildings. Peak ground acceleration measures the ground motion severity. Generally the moderate earthquake is lies between 0.1 to 0.2 g .On top floors of tall buildings acceleration is even higher and when it reaches 1 g then the building behaves as a vertical cantilever. Newton’s second law applies here that is F=M*a. When a tall building is shaken, the force acting on any part of it is still proportional to mass and acceleration but the distribution of force depends on the building the way it deforms. During horizontal shaking in all directions all walls are subjected to out of plane and in plane shear. The roof or diaphragm transfers the inertial forces to the walls and walls transfers to the ground. In case of R.C.C moment resisting frames care should be taken for the bending moments at the end of vertical members especially in case of tall buildings P-Δ effect. The stresses caused by this effect may add with earthquake effect may cause collapse of vertical members. In case of braced steel frames. The diagonal members transmit horizontal and tension forces directly and are stiffer than moment resisting frames. But the critical locations are beam column joints, these transfers the load by shear action produced by unbalanced moments at the end of members. Thus proper designing and detailing of joints is utmost importance.

1.3 STRUCTURAL PROPERTIES GOVERNING SEISMIC PERFORMANCE

1. Lateral Stiffness- stiffness of the structure is required in order to

- Reduce seismic response; this is to avoid resonance of the structure with the dominant period of the site. For taller or flexible structures rock sites are recommended.
- Control deformation; as the number of storeys increases the lateral displacement increases .This may lead to failure of non structural elements such as claddings, partitions etc.
- Influence failure modes; structures with higher stiffness undergoes less inelastic deformation.

2. Lateral Strength- The structure should have adequate strength to resist the lateral dynamic earthquake forces

3. Ductility, Hysteretic, and Energy Dissipation

Ductility is the capacity of buildings to undergo large inelastic deformations without significant strength deterioration. The graph between inertial force and displacement is known as hysteresis loop. The area enclosed by corresponding hysteresis loop is termed as energy dissipated during cycle. This loop should be stable, full and without stiffness strength degradation with more energy dissipation.

General Principles For seismic Performance

The factors which affect reliable seismic performance of structures are:

- Simplicity & Symmetry
- Stiffness
- Length in plan
- Shape in elevation
- Uniformity and continuity
- Failure modes

- Foundation conditions
- Construction materials
- Vertical irregularities

1.4 STRUCTURAL MODELLING

Earthquake response analysis is an art to simulate the behaviour of a structure subject to earthquake ground motions based on dynamics and mathematical dynamics of structure. Models should be chosen carefully keeping in mind the methods of analysis. According to Aoyama, 2001, Roesset, 1997, Gioncu, 1997and Mazzolani, 2002 modelling for frame buildings can be divided into 4 types as under-

- <u>Three dimensional models-</u> It has independent displacements at each node and can simulate any type of behaviour. These are useful to simulate the response 3-d effects such as buildings with irregular geometric configurations, torsional response in the structures with eccentric distribution of stiffness or mass, and earthquake motion in two directions or in skewed directions.

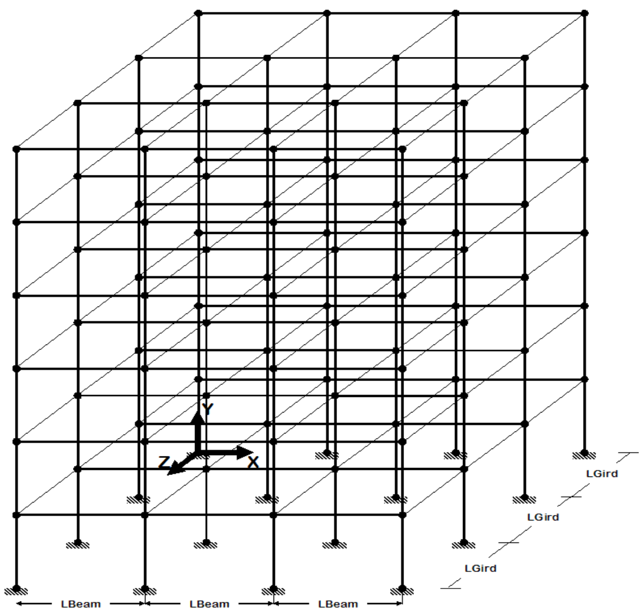


Figure No. 1.1 Three dimensional model

- <u>Two dimensional models-</u> Used for buildings that have symmetric plan and torsional response is small.In this the number of degrees of freedom gets reduced to about one fourth as compared to 3-D model.

- Lumped mass model- It is simple design for multi-storey buildings. It reduces calculations in comparison to 2-D models.

Soil interaction models- This take into account possibility of having different horizontal and vertical motions of supports, modification of natural period of structure due to soil interaction

1.5 METHODS OF ANALYSIS

After the selection of the type of model ,we can perform the analysis to determine the seismically induced forces in the structure.The type of analysis depends on external actions, behaviour of structure and the type of model.

- Linear Static Analysis-It can be used in structure with limited height only.
- Linear Dynamic Analysis- It can be response spectrum method or time history analysis. These account for higher modes of vibration and actual distribution of elastic forces in better way. Level of forces and its distribution along the height of the structure is better than static analysis.
- Non Linear Static Analysis- It allows inelastic behaviour of the structure. In this method a set of assumed static lateral incremental loads is applied over the height of the structure. This method provides information about the strength, deformation and ductility of the structure. It helps in identifying the critical members that are likely to reach limit state during earthquake. But it neglects the influence of higher modes, effect of resonance. This method is known as Push Over analysis.
- Non Linear Dynamic/ Inelastic Time History Analysis- This is the only method that describe the actual behaviour of earthquake It includes vast calculations and includes the effect of resonance , the variation of displacements at diverse levels of frame.

1.6 METHODS OF DESIGN

- Lateral Strength Design – This is based on codal provisions and is most common approach for seismic design. In this structure is assumed to possess minimum lateral strength to withstand seismic loads.
- Displacement/ Ductility Based Design – In this the structure is designed to possess adequate ductility so that it can dissipate energy and survive shocks. It is ductility based design.
- Capacity Design Methods- In this the structures are designed in such a way that hinges can form only at predetermined positions. In this method in yielding condition the strength is developed in weaker members is related to capacity of stronger member.
- Energy Based Design- It is one of the best seismic design approach that may be used in future.

1.7 DUCTILITY

Ductility is the property of material, structure to resist large inelastic deformations without significant loss of strength or stiffness. It is one of the most important factors affecting seismic performance of the structure and the gap between the actual and design lateral forces is narrowed down by providing ductility in the structure. Ductility serves as a shock absorber in a structure and reduces the transmitted force to one that is sustainable. It can also be defined as the ratio of maximum deformation that an element or structure can undergo without significant loss of initial yielding resistance to initial yield deformation. For achieving ductility building configuration should be sound. Individual members must be designed for ductility and connections and structural details should be done cautiously. To ensure that the entire structure remains ductile every structural members, joints, connections and supports should be designed with large ductility and stable hysteresis behaviour. According to Bertero, 1991 ductility is required for two main reasons first is to allow the structure as a whole, to develop its maximum potential strength, through distribution of internal forces which is given by the combination of maximum strengths of all components secondly, large structural ductility allows the structure to move as a mechanism potential strength, resulting in dissipation of large amount of energy.

Member ductility- It is the ratio of ultimate displacement to yield displacement. Yield displacement is the displacement when the load reaches yield load. Yield load is defined as the load when the reinforcement at the centre of the resultant of tensile forces in the reinforcement yields. Ultimate displacement can be defined as the maximum displacement where the load does not become lower to yield load.

Rotational and curvature ductility- The rotational ductility factor is often expressed on the basis of plastic hinge idealisation. μ_r =1+ $\frac{\theta_h}{\theta_y}$ Where θ_h= maximum plastic hinge rotation, θ_y= yield rotation in case of a beam loaded by two anti-symmetric end moments, θ_y = $\frac{M_y L}{6EI}$, where M_y ,L, I and E are yield ,moment, length,moment of inertia and modulus of elasticity of beam respectively.

Curvature ductility- It is defined as the ratio of curvature at the ultimate strength of the section to the curvature at first yield of tension steel in the section. $\mu_c = \frac{\varphi_{max}}{\varphi_y}$.

The rotational ductilities are better measure of flexural damage than curvature ductilities. It is the simple index to characterize the severity of inelastic flexure deformation.

Structural ductility- It in a global sense depends on the displacement ductility of its members because response displacement of each member can be evaluated even with static analysis.It's quantification requires a relationship between lateral loads and displacement of whole building .This may be obtained a push over analysis by plotting total base shear vs top storey displacement or preferably vs the displacement at the level where the resultant force Q_b=$\sum F_i$ is applied. The u_b is determined from the work of lateral forces F_i as follows- $u_b = \frac{\sum_{i=1}^{n} F_i u_i}{Q_b}$ where F_i is the lateral force at floor i and u_i it's lateral displacement.The code defined that the lateral force distribution can be used for analysis .The ductility of the building may be quantified by$\mu_b = \frac{\mu_{max}}{\mu_y}$.

For a single storey frame the relationship between beam ductility (μ_m) and system ductility (μ_b) is, μ_b= $\frac{\mu_m + {k_b}/{k_c}}{1+\frac{k_b}{k_c}}$ where ${k_b}/{k_c}$is the ratio of member stiffness beam and column.

In elastoplastic structure only part of the energy is returned and some portion is dissipated by plastic hinge by being converted into heat and other energies. If we consider maximum potential energy response the structure should suffer higher lateral deflection indicating requirement for ductile detailing. The I.S 1893 specifies lower lateral design forces due to earthquake taking into consideration the effect of inherent over strength and ductility provided in the structure. Therefore special requirement for ductile detailing is suggested in IS 13920.The plastic hinges, which form in structure during severe earthquake will influence curvature ductility demand in plastic hinge regions. For moment resisting frames , if yielding starts in columns before beam then it will result in column side sway mechanism and in worst case plastic hinges can form only in column of one storey making it critical leading to collapse as it may need large curvature ductility which cannot be met . And if yielding starts in beams first then beam side sway mechanism may develop which may require moderate curvature ductility demands. So therefore mixed side sway demand can be met by careful detailing. The rational approach for seismic resistance is to take the most suitable mechanism of post elastic deformation for the structure and suitable design procedure so that yielding occurs in predetermined manner during severe earthquake. One of the best methods to achieve post elastic deformation is by flexural yielding at selected plastic hinge positions in a chosen collapse mechanism for moment resisting frames as by this plastic hinges can be made adequately ductile.

Factors Affecting Ductility-

- Ductility increases linearly with an increase in shear strength carried by concrete for small value of axial compressive stress.
- Ductility linearly reduces up to the point where axial compressive stress becomes equal to the axial compressive stress at balanced failure.
- The confining of concrete increases the ductility as it increases with increase in ultimate strength of concrete
- Ductility decreases with an increase in yield strength of steel.
- The shear failure in section can be prevented by providing lateral reinforcement.

- The presence of an enlarged compression flange in a T beam reduces the depth of the compression zone at collapse and thus increases ductility.

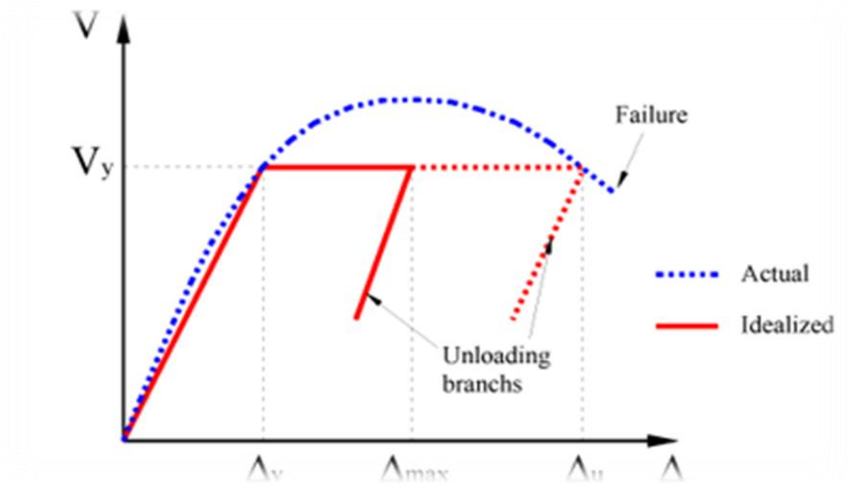


Figure No. 1.2 Graph Base shear Vs displacement graph

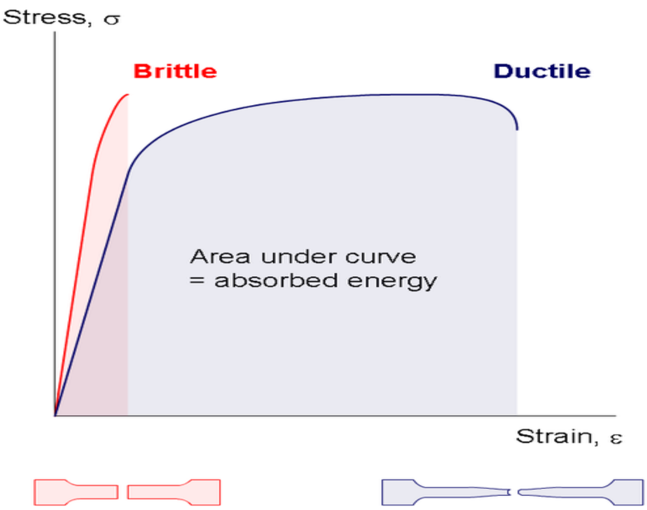


Figure No.1.1 Stress- strain graph for brittle and ductile materials

<u>Candidates for ductile detailing</u>

According to IS1893-1984 structures should be detailed for ductility if they satisfy one or more conditions-

- The structure is located in seismic zone IV or V;
- The structure is located in seismic zone III and has the importance factor (I) greater than 1.0;
- The structure is located in seismic zone III and is an industrial structure; and
- The structure is located in seismic zone III and is more than 5 storey high.

CHAPTER 2

LITERATURE

2.1 METHODS OF ANALYSIS

For seismic performance evaluation, a structural analysis of the mathematical model of the structure is required to determine force and displacement demands in various components of the structure. Several analysis methods, both elastic and inelastic, are available to predict the seismic performance of the structures. (sermin, 2005)

2.1.1 ELASTIC METHODS OF ANALYSIS

The force demand on each component of the structure is obtained and compared with available capacities by performing an elastic analysis. Elastic analysis methods include code static lateral force procedure, code dynamic procedure and elastic procedure using demand capacity ratios. These methods are also known as force-based procedures which assume that structures respond elastically to earthquakes. In code static lateral force procedure, a static analysis is performed by subjecting the structure to lateral forces obtained by scaling down the smoothened soil-dependent elastic response spectrum by a structural system dependent force reduction factor, "R". In this approach, it is assumed that the actual strength of structure is higher than the design strength and the structure is able to dissipate energy through yielding. In code dynamic procedure, force demands on various components are determined by an elastic dynamic analysis. The dynamic analysis may be either a response spectrum analysis or an elastic time history analysis. Sufficient number of modes must be considered to have a mass participation of at least 90% for response spectrum analysis. Any effects of higher modes are automatically included in time history analysis. In demand/capacity ratio (DCR) procedure, the force actions are compared to corresponding capacities as demand/capacity ratios. Demands for DCR calculations must include gravity effects. While code static lateral force and code dynamic procedures reduce the full earthquake demand by an R-factor, the DCR approach takes the full earthquake demand without reduction and adds it to the gravity demands. DCRs approaching 1.0 (or higher) may indicate potential deficiencies. Although force-based procedures are well known by engineering profession and easy to apply, they have certain drawbacks. Structural components are evaluated for serviceability in the elastic range of strength and deformation. Post-elastic behaviour of structures could not be identified by an elastic analysis. However, post-elastic behaviour should be considered as almost all structures are expected to deform in inelastic range during a strong earthquake. The seismic force reduction factor "R" is utilized to account for inelastic behaviour indirectly by reducing elastic forces to inelastic. Force reduction factor, "R", is assigned considering only the type of lateral system in most codes, but it has been shown that this factor is a function of the period and ductility ratio of the structure as well. Elastic methods can predict elastic capacity of structure and indicate where the first yielding will occur, however they don't predict failure mechanisms and account for the redistribution of forces that will take place as the yielding progresses. Real deficiencies present in the structure could be missed. Moreover, force-based methods primarily provide life safety but they can't provide damage limitation and easy repair. The drawbacks of force-based procedures and the dependence of damage on deformation have led the researches to develop displacement-based procedures for seismic performance evaluation. Displacement-based procedures are mainly based on inelastic deformations rather than elastic forces and use nonlinear analysis procedures considering seismic demands and available capacities explicitly. (sermin, 2005)

2.1.2 INELASTIC METHODS OF ANALYSIS

Structures suffer significant inelastic deformation under a strong earthquake and dynamic characteristics of the structure change with time so investigating the performance of a structure requires inelastic analytical procedures accounting for these features. Inelastic analytical procedures help to understand the actual behaviour of structures by identifying failure modes and the potential for progressive collapse. Inelastic analysis procedures basically include inelastic time history analysis and inelastic static analysis which is also known as pushover analysis.

The inelastic time history analysis is the most accurate method to predict the force and deformation demands at various components of the structure. However, the use of inelastic time history analysis is limited because dynamic response is very sensitive to modelling and ground motion characteristics. It requires proper modelling of cyclic load-deformation characteristics considering deterioration properties of all important components. Also, it requires availability of a set of representative ground motion records that accounts for uncertainties and differences in severity, frequency and duration characteristics. Moreover, computation time, time required for input preparation and interpreting voluminous output make the use of inelastic time history analysis impractical for seismic performance evaluation. Inelastic static analysis, or pushover analysis, has been the preferred method for seismic performance evaluation due to its simplicity. It is a static analysis that directly incorporates nonlinear material characteristics. Inelastic static analysis procedures include Capacity Spectrum Method, Displacement Coefficient Method and the Secant Method. (sermin, 2005).

2.2 INTRODUCTION TO PUSHOVER ANALYSIS

One of the emerging fields in seismic design of structures is the Performance Based Design. The subject is still in the realm of research and academics, and is only slowly emerging out into the practitioner's arena. Seismic design is slowly transforming from a stage where a linear elastic analysis for a structure was sufficient for both its elastic and ductile design, to a stage where a specially dedicated non-linear procedure is to be done, which finally influences the seismic design as a whole. The basis for the linear approach lies in the concept of the Response Reduction factor R. When a structure is designed for a Response Reduction factor of, say, R = 5, it means that only 1/5th of the seismic force is taken by the Limit State capacity of the structure. Further deflection is in its ductile behaviour and is taken by the ductile capacity of the structure. In Reinforced Concrete (RC) structures, the members (ie., beams and columns) are detailed such as to make sure that the structure can take the full impact without collapse beyond its Limit State capacity up to its ductile capacity. In fact we never analyse for the ductile part, but only follow the reinforcement detailing guidelines for the same. The drawback is that the response beyond the limit state is neither a simple extrapolation, nor a perfectly ductile behaviour with predeterminable deformation capacity. This is due to various reasons: the change in stiffness of members due to cracking and yielding, P-delta effects, change in the final seismic force estimated, etc. Although elastic analysis gives a good indication of elastic capacity of structures and shows where yielding might first occur, it cannot account for redistribution of forces during the progressive yielding that follows and predict its failure mechanisms, or detect possibility and location of any premature failure. A non-linear static analysis can predict these more accurately since it considers the inelastic behaviour of the structure. It can help identify critical members likely to reach critical states during an earthquake for which attention should be given during design and detailing.

The need for a simple method to predict the non-linear behaviour of a structure under seismic loads saw light in what is now popularly known as the Pushover Analysis (PA). It can help demonstrate how progressive failure in buildings really occurs, and identify the mode of final failure. Putting simply, Pushover analysis is a non-linear analysis procedure to estimate the strength capacity of a structure beyond its elastic limit(meaning Limit State) up to its ultimate strength in the post-elastic range. In the process, the method alsopredicts potential weak areas in the structure, by keeping track of the sequence of damages of each and every member in the structure (by use of what are called ‘hinges’ they hold).

2.3 PUSHOVER ANALYSIS EXPLAINED

- Pushover is a static-nonlinear analysis method where a structure is subjected to gravity loading and a monotonic displacement-controlled lateral load pattern which continuously increases through elastic and inelastic behaviour until an ultimate condition is reached.

- Pushover analysis can be performed as force-controlled or displacement controlled. In force-controlled pushover procedure, full load combination is applied as specified, i.e, force-controlled procedure should be used when the load is known (such as gravity loading).In displacement-controlled procedure, specified drifts are sought (as in seismic loading) where the magnitude of applied load is not known in advance.

- One of the fundamental simplifications underlying the concept of Pushover analysis is that it considers the structure as a single degree of freedom (SDOF) system, which in reality it hardly is. And that means the structure model, with numerous joints with lumped masses, is assumed to be equivalent to a single vertical strut fixed at bottom with a single (but considerable) mass lumped at the top.

- The two important terms are static and analysis Static implies that a static method has been employed to represent a dynamic phenomenon which may be adequate in many cases but may be doomed to failure in some cases. Analysis implies that a solution has been created already and push over has been employed to evaluate the solution and modify it as needed. Thus push over is a part of evaluation process and provides estimates of the demands imposed on the structure in a rational and in efficient way.

2.4 NEED FOR PUSHOVER ANALYSIS

- Pushover Analysis in the recent years is becoming a popular method of predicting seismic forces and deformation demands for the purpose of performance evaluation of existing and new structures.
- Pushover analysis is a partial and relatively simple intermediate solution to the complex problem of predicting force and deformation demands imposed on structures and their elements by severe ground motion. Pushover analysis is one of the analysis methods recommended by Eurocode and FEMA 273 ,FEMA440
- Pushover analysis provides valuable insights on many response characteristics like • Force Demand on Potentially brittle elements. • Consequences of strength deterioration of individual elements on structural behavior. • Identification of critical regions in which the deformation demands are expected to be high and that have to become the focus of through detailing. • Identification of strength discontinuities in plan or elevation that will lead to changes in dynamic characteristics in the inelastic region. • Verification of completeness and adequacy of load path, considering all structural and non structural elements of the structural system.

2.5 SIMILARITIES IN PUSHOVER ANALYSIS AND CONVENTIONAL ANALYSIS

- Both Static and Pushover Analysis apply lateral load of a predefined vertical distribution pattern on the structure. In SA, the lateral load is distributed either parabolically (in Seismic Coefficient method) or proportional to the modal combination (in the direct combination method of Response Spectrum). In PA, the distribution is proportional to height raised to the power of ‘k’, where k (equivalent to ‘2’ in the equation under Cl. 7.7.1 in IS:1893-2002) can be equal to 0 (uniform distribution), 1 (the inverted triangle distribution), 2 (parabolic distribution as in the seismic coefficient method) or a calculated value between 1 and 2, the value of k being based on the time period T of the structure, as per the FEMA 356 (where k is given a value of 2 if $T \geq 2.5$ seconds, a value of 1 if $T \leq 0.5$ seconds and interpolated for intermediate values of T). The distribution can also be proportional to either the first mode shape, or a combination of modes.
- In both Static and Pushover Analysis, the maximum lateral load estimated for the structure is calculated based on the fundamental time period of the structure.

2.6 STATIC ANALYSIS VS PUSHOVER ANALYSIS

- While in SA the initial time period is taken to be a constant (equal to its initial value), in PA this is continuously re-calculated as the analysis progresses.
- SA uses an elastic model, while PA uses a non-linear model. In the latter this is incorporated in the form of non-linear hinges inserted into an otherwise linear elastic model which one generates using a common structural analysis & design software package (like SAP2000 or STAAD.Pro), having facilities for Pushover Analysis.

2.7 THE METHOD

In general, it is the method of analysis by applying specified pattern of direct lateral loads on the structure, starting from zero to a value corresponding to a specific displacement level, and identifying the possible weak points and failure patterns of a structure. The performance of the structure is evaluated using the status of hinges at target displacement or performance point corresponding to specified earthquake level (the given response spectrum). The performance is satisfactory if the demand is less than capacity at all hinge locations. As the loading and evaluation procedures are only virtually correct with respect to the real earthquake events, it differs from the rigorous dynamic analysis in many ways.

2.8 LIMITATIONS OF PUSHOVER ANALYSIS

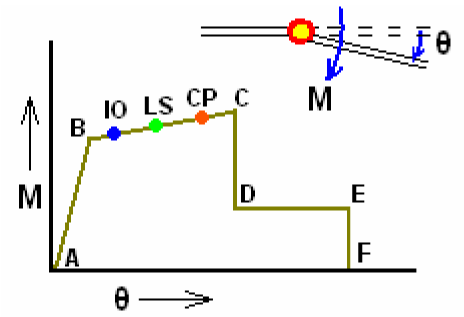
- Static pushover analysis neglects dynamic effects. Hence, during an earthquake, the inelastic structural behaviour can be described by balancing the dynamic equilibrium at every time step. As pushover analysis focuses only on the strain energy of the structure during a monotonic static push, it neglects other sources of energy mainly associated with dynamic components of forces such as kinetic energy and viscous damping energy.

- However, pushover analysis is more appropriate for low to mid-rise buildings with dominant fundamental mode response. For special and high-rise buildings, pushover analysis should be complemented with other evaluation procedures since higher modes could certainly affect the response.

2.9 HINGE

Hinges are points on a structure where one expects cracking and yielding to occur in relatively higher intensity so that they show high flexural (or shear) displacement, as it approaches its ultimate strength under cyclic loading. These are locations where one expects to see cross diagonal cracks in an actual building structure after a seismic mayhem, and they are found to be at the either ends of beams and columns, the 'cross' of the cracks being at a small distance from the joint – that is where one is expected to insert the hinges in the beams and columns of the corresponding computer analysis model. Hinges are of various types– namely, flexural hinges, shear hinges and axial hinges. The first two are inserted into the ends of beams and columns. Since the presence of masonry infills have significant influence on the seismic behaviour of the structure, modelling them using equivalent diagonal struts is common in PA, unlike in the conventional analysis, where its inclusion is a rarity. The axial hinges are inserted at either ends of the diagonal struts thus modelled, to simulate cracking of

infills during analysis. Basically a hinge represents localised force-displacement relation of a member through its elastic and inelastic phases under seismic loads. For example, a flexural hinge represents the moment-rotation relation of a beam of which a typical one is as represented in Fig. AB represents the linear elastic range from unloaded state A to its effective yield B, followed by an inelastic but linear response of reduced (ductile) stiffness from B to C. CD shows a sudden reduction in load resistance, followed by a reduced resistance from D to E, and finally a total loss of resistance from E to F. Hinges are inserted in the structural members of a framed structure typically as shown in Fig.2. These hinges have non-linear states defined as ‘Immediate Occupancy’ (IO), ‘Life Safety’ (LS) and ‘Collapse Prevention’ (CP) within its ductile range. This is usually
done by dividing B-C into four parts and denoting IO, LS and CP, which are states of each individual hinges(in spite of the fact that the structure as a whole too have these states defined by drift limits). There are different criteria for dividing the segment BC. For instance, one such specification is at 10%, 60%, and 90%of the segment BC for IO, LS and CP respectively (Inel & Ozmen, 2006).



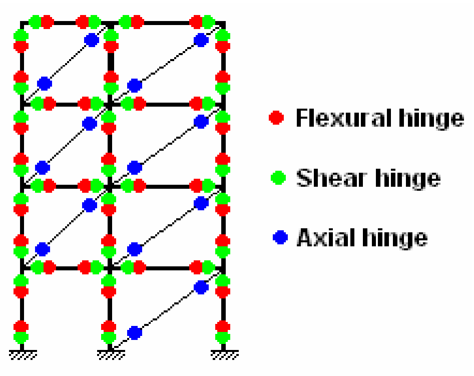


Fig2.1: A Typical Flexural Hinge Property, showing in IO (Immediate Occupancy), LS (Life Safety) and CP (Collapse Prevention)

Fig.2.2: Typical Locations of Hinges structural Model

2.10 TWO STAGE DESIGN APPROACH

Although hinge properties can be obtained from charts of average values included in FEMA356,ATC-40 and FEMA 440 (which are only rough estimates), for accurate results one requires the details of reinforcement provided in order to calculate exact hinge properties (using concrete models such as the Confined Mander model available in the SAP2000 software package). And one has to design the structure in order to obtain the reinforcement details. This means that PA is meant to be a second stage analysis. Thus the emerging methodology to an accurate seismic design is: (1) first a linear seismic analysis based on which a primary structural design is done; (2) insertion of hinges determined based on the design and then (3) a pushover analysis, followed by (4) modification of the design and detailing, wherever necessary, based on the latter analysis.

- On Static analysis, the analysis results are always the elastic (limit state) forces (moment, shear and axial forces) to be designed for. In PushoverAnalysis, in the global sense, it is the base shear (Vb) vs roof top displacement (Δroof top, taken as displacement of a point on the roof, located in plan at the centre of mass), plotted up to the termination of the analysis. At a local level, it is the hinge states to be examined and decided on the need for its redesign or a retrofit.

Pushover Analysis can be useful under two situations: When an existing structure has deficiencies in seismic resisting capacity (due to either omission of seismic design when built, or the structure becoming seismically inadequate due to a later up gradation of the seismic codes) is to be retrofitted to meet the present seismic demands, Pushover Analysis can show where the retrofitting is required and how much. In fact this was what Pushover Analysis was originally developed for, and for which it is still widely used. For a building in its design phase, Pushover Analysis results help scrutinise and fine tune the seismic design based on Static Analysis, which is slowly becoming more of a standard procedure for large critical structures.

- Static Analysis, being a linear analysis, is done independently for dead and live loads, and the results added up to give the design forces. But since PA is non-linear, the gravity loads and the lateral load cases are applied sequentially in a single analysis.
- In Static Analysis, the loads are factored, since the results are for the design, but since Pushover Analysis is done to simulate the behaviour under actual loads, the loads applied are not factored. Thus in a Pushover Analysis, the gravity loads are applied in accordance with Cl.7.3.3 and Table 8 of IS:1893-2002, giving a combination of [DL +0.25 LL(≤3kN/sq.m) + 0.5 LL(>3kN/sq.m)] – where DL denotes Dead Loads and LL, Live Load.
- In Static Analysis, the lateral load of a calculated intensity is applied in whole – in one shot. In Pushover analysis, structure model (ie, the computer model for analysis) is gently ‘pushed over’ by a monotonically increasing lateral load, applied in steps up to a predetermined value or state.
- This predetermined value or state depends on the method used. One is the Displacement Coefficient Method (DCM) of FEMA 356, where a Target Displacement is calculated to which the structure is ‘pushed’. Eurocode 8 (EN 1998-1, 2003) also follows more or less the same approach. The other is the Capacity Spectrum Method (CSM) of ATC-40, where the load is incremented and checked at each stage, until what is called the ‘Performance Point’ condition is reached. FEMA 440 presents improvements in the procedure of both these methods. In this article, only the CSM (as described in ATC-40) is dealt with, since it is found to be more suitable than DCM for RC structures.

2.11 PROCEDURES AT A GLANCE

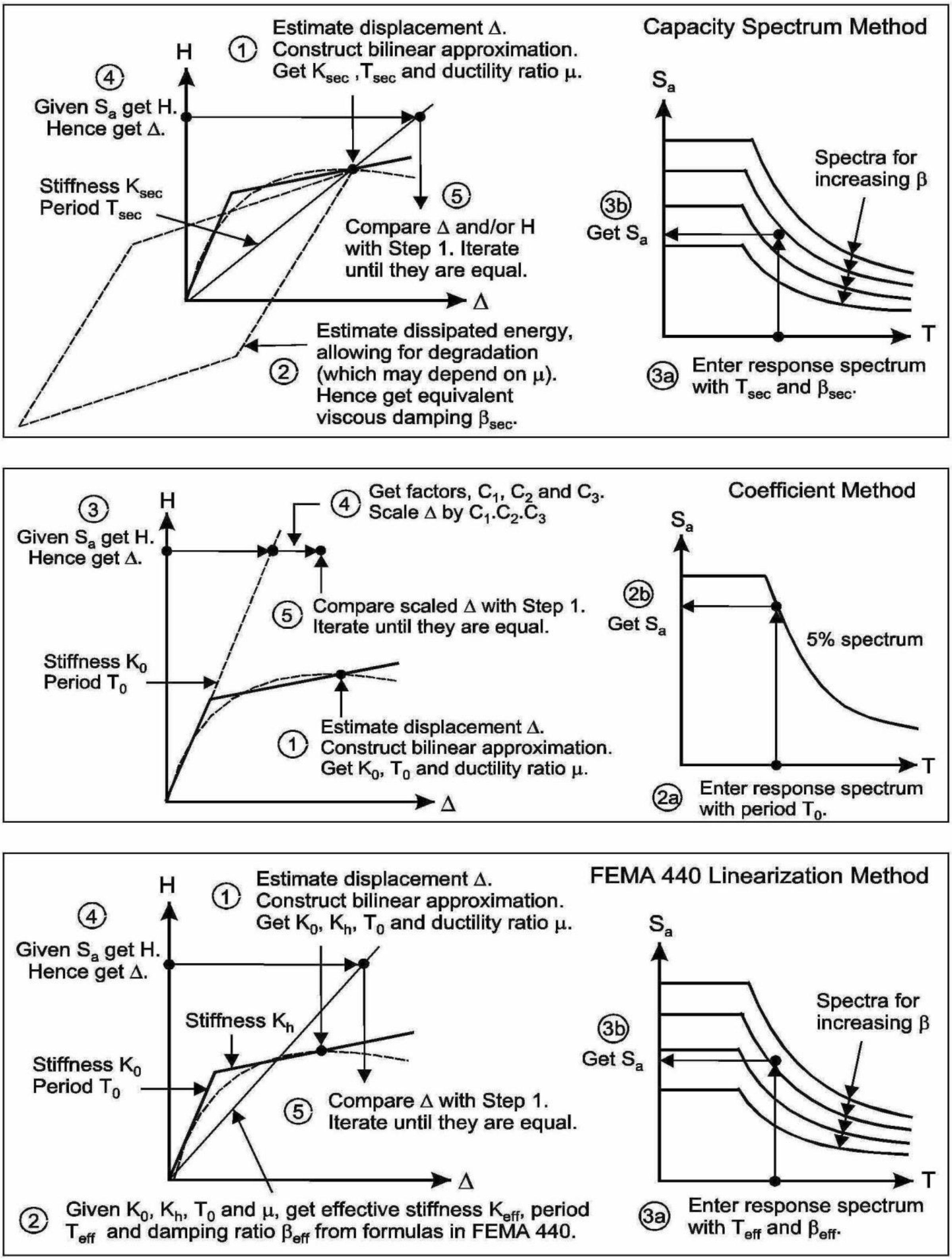


Figure No. 2.3 Different procedures at a glance

Although the procedures for building evaluation are different from one another, their basic principles are all the same and they all use the bilinear approximation of the pushover curve. This static procedure equates the properties of every Multi degree of freedom (MDOF) structures to corresponding Single degree of freedom (SDOF) equivalents, and approximates the expected maximum displacement using the Response spectrum of relevant earthquake intensity.

- ***ATC 40 - 1996 - Capacity Spectrum Method (CSM)*** This method is based on the equivalent linearization of a nonlinear system. The important assumption here is that inelastic displacement of a nonlinear SDF system will be approximately equal to the maximum elastic displacement of linear SDF system with natural time period and damping values greater than the initial values for those in nonlinear system. ***ATC 40*** describes three procedures (A,B and C) for the ***CSM*** .
- ***FEMA 356 - 2000 - Displacement Coefficient Method(DCM)*** Here, the nonlinear MDF system's displacement is obtained from the linear elastic demand spectrum, using certain coefficients which are based on empirical equations derived by calibration against a large number of dynamic analyses.
- ***FEMA 440 - 2005 - Equivalent Linearization - Modified CSM*** This improved version of equivalent linearization is derived from the statistical analysis of large number of responses against different earthquake ground motions. The assumption in CSM that the equivalent stiffness of inelastic system will be the same as its secant stiffness is not used here. Instead, the equivalent stiffness is obtained from effective time period and damping properties derived using equations from statistical analyses.
- ***FEMA 440 - 2005- Displacement Modification- Improvement for DCM*** This improvement for the earlier Displacement coefficient method uses advanced equations for different coefficients. Coefficient for P $-\Delta$ effects is replaced with a lateral dynamic instability check by defining a maximum value of lateral strength R, such that

  $Rmax = \Delta d/\Delta y + (\alpha e)^{\wedge} -t/4$

  where, the terms are as described below:

  $\Delta d$and Δy are the displacements corresponding to maximum base shear *Vd* and effective yield strength *Vy* respectively

  If *Ke* is the effective stiffness of the building, which is the slope of the line joining zero base shear point and the point at 60% of idealized yield strength, obtained from idealization of pushover curve in to linear portions,

  $\alpha 1$ *Ke* = effective post yield stiffness with positive slope,

  $\alpha 2$ *Ke* = maximum (negative) post -elastic stiffness, which is the slope of the line connecting points of maximum base shear and 60% yield strength on the post-elastic curve,

  $a_{p-\Delta}$ *Ke* = Slope of the tangent at the point of maximum base shear,

  αe *Ke* = effective post elastic (negative) stiffness,

  Where, $\alpha_e = \alpha_{p-\Delta} + (a_2 - \alpha_{p-\Delta})$

λ, a factor representing ground motion effects, = 0.2 for far field motions and 0.8 for near field motions

If T = fundamental time period of the building, t = 1+ 0.15 $ln\ T$

$$R=\frac{S_a}{V_{Y/W}}C_m$$

Where, Vy = Yield strength calculated using results of the pushover analysis for the idealized nonlinear force displacement curve,

Sa = Spectral acceleration obtained from the demand spectrum with specified damping, corresponding to the effective time period Te, obtained from the idealized pushover curve,

W = Effective seismic weight of the building including the total dead load and applicable portions of other gravity loads as given in FEMA 356, and

Cm = Effective mass factor which is taken as the effective modal mass for 1st mode of the structure

ASCE 41-06

ASCE 41-06: Seismic Rehabilitation of Buildings (2006) serves to provide a standard for nationally applicable provisions in the seismic rehabilitation of existing buildings and supersedes the previous standards; FEMA 273: NEHRP Guidelines for the Seismic Rehabilitation of Buildings (1997) and FEMA 356: Prestandard and Commentary for Seismic Rehabilitation of Buildings (2000). While ATC 14 (1987) created the concept of screening buildings for potential deficiencies, FEMA 273 (1997) was the first standard that provided ‘‘displacement based’’ methodologies for nonlinear analysis of all types of structures. Prior to those documents, seismic evaluation and retrofit was primarily depended on to the judgment of the design professional by using the standards for new building design to evaluate and retrofit existing buildings.

ASCE 41-06 defines seismic rehabilitation by improving the seismic performance of structural and/or nonstructural components of a building by correcting deficiencies identified in a seismic evaluation. Unlike ASCE 7, which employs ‘‘force-based’’ procedures by utilizing a global building ductility factor (R-factor), ASCE 41 uses ‘‘displacement-based’’ procedures which assess the ductility of each element action (shear, flexure, etc.) individually. Also, ASCE 41 contains specific guidance on the use of nonlinear analysis procedures which ASCE 7 doesn’t contain. ASCE 41 can also be used to rehabilitate the historic structures where performance based rehabilitations are desired. If seismic upgrading interest is found after defined methodology to identify the deficiencies, several considerations should be studied such as; structural characteristics, site seismic hazards, results from prior seismic evaluations, historic status, economic considerations and societal issues. Economical considerations are proven to be one of the most decisive aspects for whether a retrofit consideration goes from planning to implementation. After the initial phase, if the rehabilitation project is decided to be done, rehabilitation must be done in accordance with target building performance level, earthquake hazard level and rehabilitation objective classification. ASCE 41-06 and FEMA 356 defines six structural performance levels expected for post-earthquake state shown in Table 3.1 below:

- Immediate Occupancy (S1)
- Damage Control Range (S2)
- Life Safety (S3)
- Limited Life Safety Range (S4)
- Collapse Prevention (S5)
- Not Considered (S6)

Target building performance and levels from ASCE 41-06 Table C1-8

	Structural Performance Levels and Ranges					
Nonstructural Performance Levels	Immediate Occupancy (S-1)	Damage Control Range (S-2)	Life Safety (S-3)	Limited Safety Range (S-4)	Collapse Prevention (S-5)	Not Considered (S-6)
Operational (N-A)	Operational 1-A	2-A	Not recommended	Not recommended	Not recommended	Not recommended
Immediate Occupancy (N-B)	Immediate Occupancy 1-B	2-B	3-B	Not recommended	Not recommended	Not recommended
Life Safety (N-C)	1-C	2-C	Life Safety 3-C	4-C	5-C	6-C
Hazards Reduced (N-D)	Not recommended	2-D	3-D	4-D	5-D	6-D
Not Considered (N-E)	Not recommended	Not recommended	Not recommended	4-E	Collapse Prevention 5-E	Not rehabilitation

Figure No. 2.4 Structural performance and Ranges

Operational Occupancy Performance Level: The post-earthquake damage to the structure is very light. There is no permanent building drift. The structure maintains its original strength and stiffness. There is very little damage. The backup building services maintain function.

Immediate Occupancy Performance Level: The post-earthquake damage to the structure is light. There is no permanent building drift. The structure maintains most of its original strength and stiffness. The risk to life threatening injury from structural damage is very low. Some minor repairs may be appropriate, but are not required for re-occupancy.

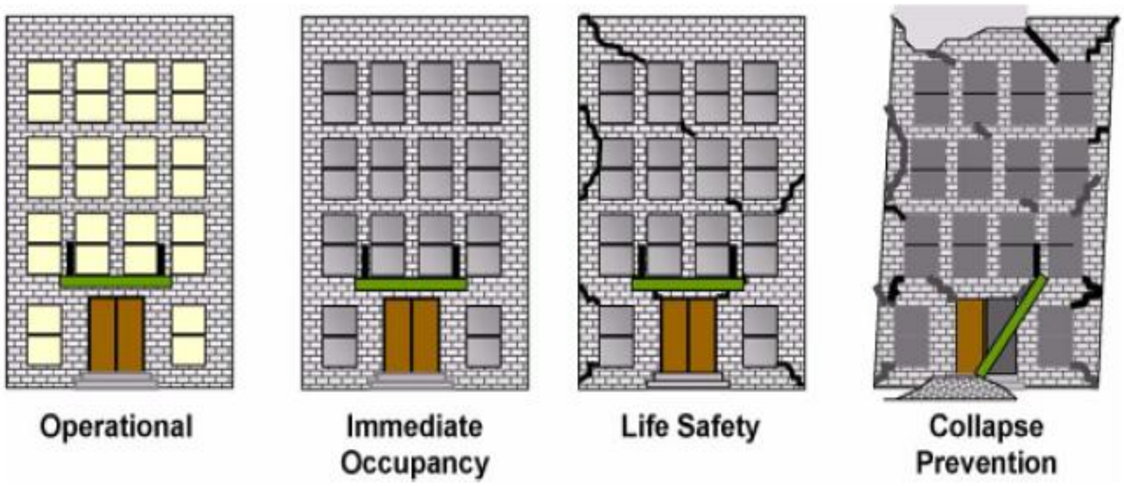
Life Safety Performance Level: The post-earthquake damage to the structure is significant, but some margin against either partial or total structural collapse remains.

Some structural elements and components are severely damaged but this has not resulted in large falling debris hazards, either inside or outside the building. Injuries may occur during the earthquake; however the overall risk of life threatening injury as a result of structural damage is expected to be low. It should be possible to repair the structure; however for economic reasons this may not be practical. Although the damaged structure is not an imminent collapse risk, it would be prudent to implement structural repairs or install temporary bracing prior to reoccupying the building.

Collapse Prevention Performance Level: The post-earthquake damage is so significant that the building is on the verge of partial or total collapse. Substantial damage to the structure has occurred, potentially including significant degradation in the stiffness and strength of the lateral-force-resisting system, large permanent lateral deformation of the structure, and (to a limited extent) degradation in vertical load carrying capacity. However, all significant components of the gravity load resisting system must continue to carry their gravity loads.The structure may not be technically practical to repair and is not safe to reoccupy.

It should be noted, that Immediate Occupancy and Operational Performance Levels are very costly and typically not practical for structures unless they are needed to maintain their service after an earthquake like hospital, police stations, etc.

The expected post-earthquake damage states are shown in the Figure



Expected post-earthquake damage states from ASCE 41-06 (Courtesy of R.Hamburger)

Figure No. 2.5 Performance and structural deformation demand for ductile systems.

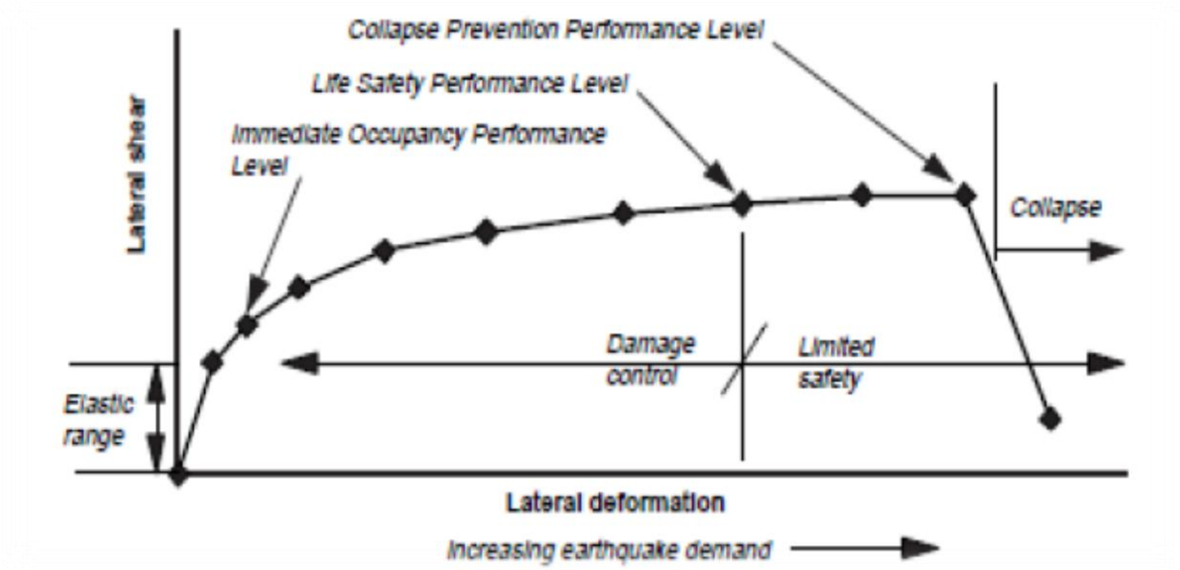


Figure No. 2.6 Lateral deformation VS Lateral shear graph

2.12 EVALUATION OF SEISMIC PERFORMANCE

The seismic performance of a building can be evaluated in terms of pushover curve, performance point, displacement ductility, plastic hinge formation etc. The base shear vs. roof displacement curve is obtained from the pushover analysis from which the maximum base shear capacity of structure can be obtained. This capacity curve is transformed into capacity spectrum by ETABS as per ATC40 and demand or response spectrum is also determined for the structure for the required building performance level. The intersection of demand and capacity spectrum gives the performance point of the structure analyzed.

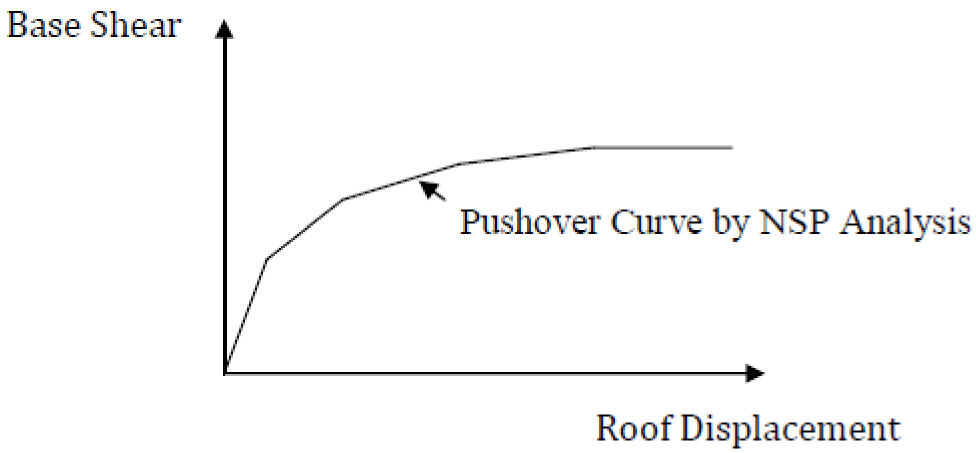


Figure No. 2.7 Base shear vs. Roof displacement

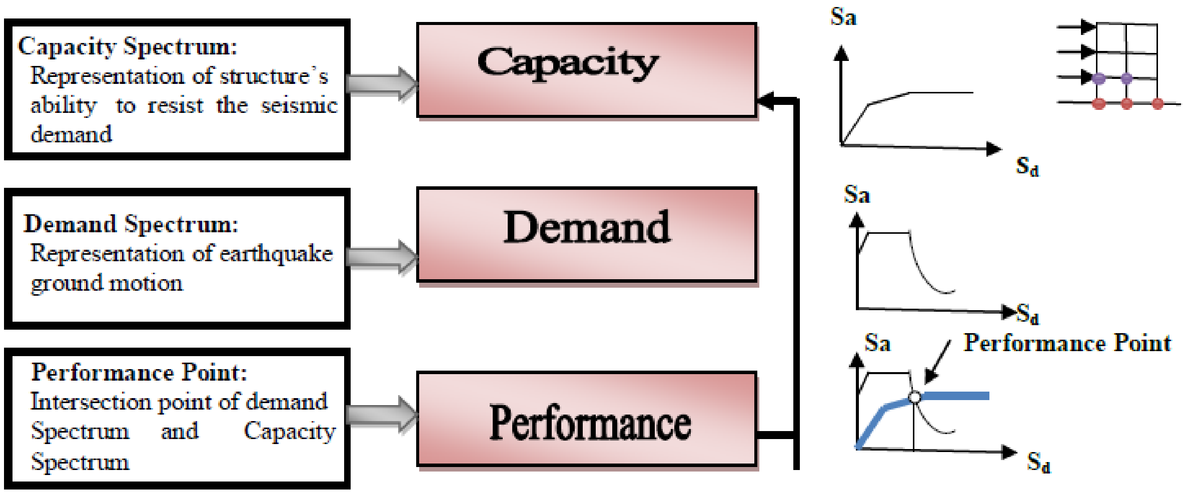


Figure No. 2.8 Performance point

2.13 DETERMINATION OF PERFORMANCE POINT

According to ATC 40

There are three procedures described in ATC-40 to find the performance point.

- ***Procedure A***, which uses a set of equations described in ATC-40.
- ***Procedure B*** is also an iterative method to find the performance point, which uses the assumption that the yield point and the post yield slope of the bilinear representation, remains constant. This is adequate for most cases; however, in some cases this assumption may not be valid.
- ***Procedure C*** is graphical method that is convenient for hand as well as software analysis. ETABS uses this method for the determination of performance point. To find the performance point using Procedure C the following steps are used:

First of all, the single demand spectrum (variable damping) curve is constructed by doing the following for each point on the Pushover Curve:

1) Draw a radial line through a point **(P)** on the Pushover curve. This is a line of constant period.

2) Calculate the damping associated with the point **(P)** on the curve, based on the area under the curve up to that point.

3) Construct the demand spectrum, plotting it for the same damping level as associated with the point 'P' on the pushover curve.

4) The intersection point (P') for the radial line and associated demand spectrum represents a point on the Single Demand Spectrum (Variable Damping Curve).

A number of arbitrary points are taken on the Pushover curve. A curve is then drawn by joining through these points. The intersection of this curve with the original pushover curve gives the performance point of the structure as shown in Figure

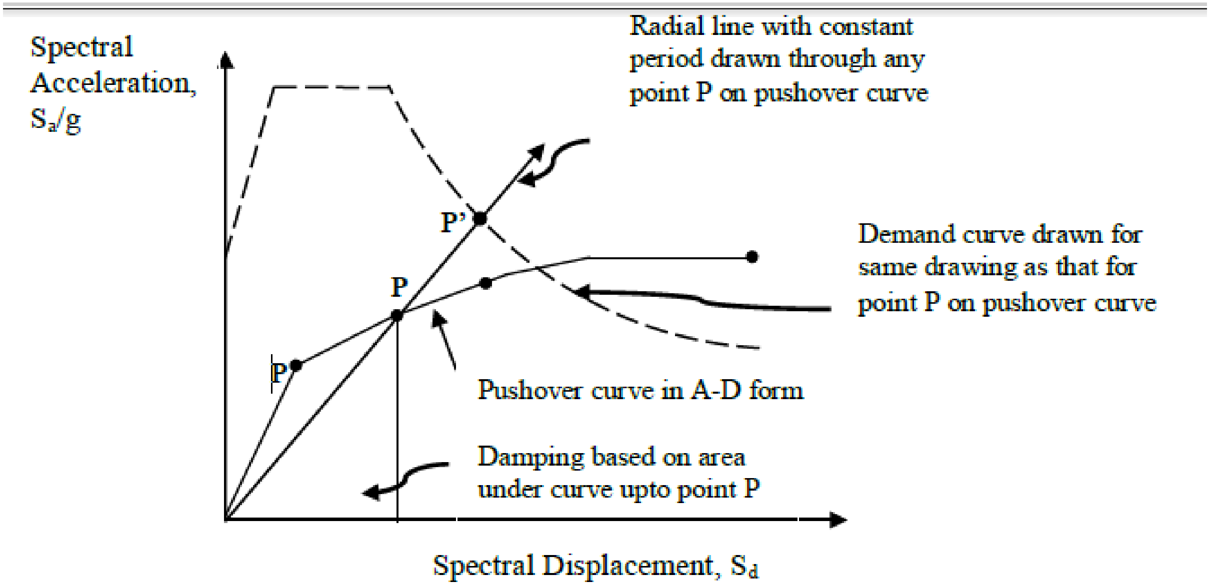


Figure No. 2.9 Capacity Spectrum Procedure 'C' to Determine Performance Point

The sequence of plastic hinge formation and state of hinge at various levels of building performance can be obtained from ETABS output. This gives the information about the weakest member. Accordingly the detailing of the member can be done in order to achieve the desired pattern of failure of members in case of severe earthquakes. It is concluded that pushover analysis is a successful method in determination of the sequence of yielding of the components of a building, possible mode of failure, and final state of the building after a predetermined level of lateral load is sustained by the structure.

2.14 ASSUMPTIONS

Following assumptions are made while analyzing a structure in the ETABS:

(i) The material is homogeneous, isotropic,
(ii) All columns supports are considered as fixed at the foundation,
(iii) Tensile strength of concrete is ignored in sections subjected to bending,
(iv) The super structure is analyzed independently from foundation and soil medium, on the assumptions that foundations are fixed,
(v) Pushover hinges are assigned to all the member ends. In case of Columns PMM hinges (i.e. Axial Force and Biaxial Moment Hinge) are provided while in case of beams M3 hinges (i.e. Bending Moment hinge) are provided,
(vi) The maximum target displacement of the structure is calculated in accordance with the guidelines given by FEMA 356 for maximum roof level lateral drift. Performance of building has been classified into 5 levels, viz. (i) Operational

(OP), (ii) Immediate Occupancy (IO), (iii) Damage Control (DC), (iv) Life Safety (LS) and (v) Collapse Prevention (CP).

2.15 The Target displacement

Target displacement is the displacement demand for the building at the control node subjected to the ground motion under consideration. This is a very important parameter in pushover analysis because the global and component responses (forces and displacement) of the building at the target displacement are compared with the desired performance limit state to know the building performance. So the success of a pushover analysis largely depends on the accuracy of target displacement.

There are two approaches to calculate target displacement: (a) Displacement Coefficient Method (DCM) of FEMA 356 and (b) Capacity Spectrum Method (CSM) of ATC 40. Both of these approaches use pushover curve to calculate global displacement demand on the building from the response of an equivalent single-degree-of-freedom (SDOF) system. The only difference in these two methods is the technique used

According to Displacement coefficient method of FEMA 356

is calculated by dt = C0C1C2C3 SaTe ^2g/4p^2 where:

- C0 = Modification factor for SDOF to MDOF
- C1 = Modification Factor to relate expected maximum inelastic displacements to displacements calculated for liner elastic response
- C2 = Modification factor to represent the effect of hysteresis shape on the maximum displacement response
- C3 = Modification Factor to represent increased displacements due to dynamic P-Δ effects. Sa = Response spectrum acceleration
- Te = Characteristic period of the response spectrum.

According to Capacity Spectrum Method ATC 40

The basic assumption in Capacity Spectrum Method is also the same as the previous one. That is, the maximum inelastic deformation of a nonlinear SDOF system can be approximated from the maximum deformation of a linear elastic SDOF system with an equivalent period and damping. This procedure uses the estimates of ductility to calculate effective period and damping. This procedure uses the pushover curve in an acceleration-displacement response spectrum (ADRS) format. This can be obtained through simple conversion using the dynamic properties of the system. The pushover curve in an ADRS format is termed a ‘capacity spectrum’ for the structure. The seismic ground motion is represented by a response spectrum in the same ADRS format and it is termed as demand spectrum

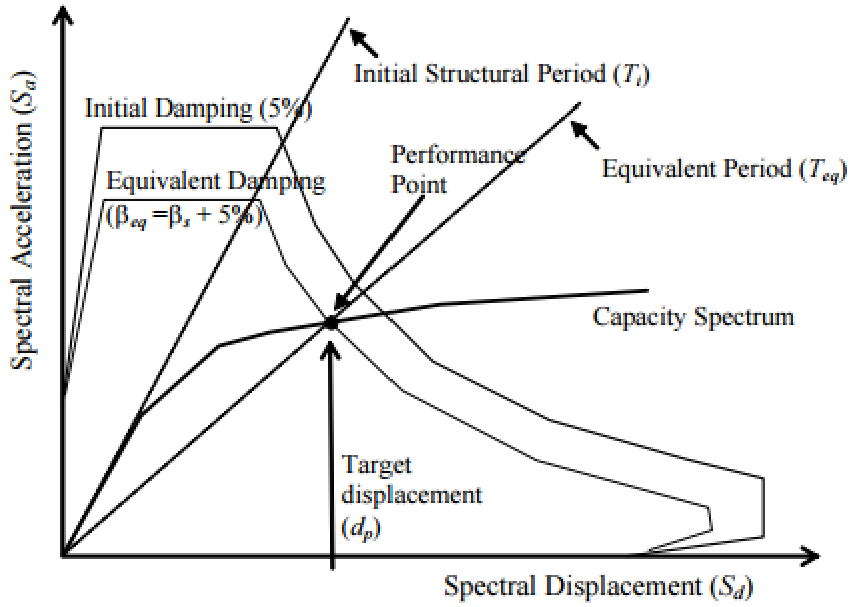


Figure No. 2.10: Schematic representation of Capacity Spectrum Method (ATC 40)

The equivalent period (Teq) is computed from the initial period of vibration (Ti) of the nonlinear system and displacement ductility ratio (μ). Similarly, the equivalent damping ratio (βeq) is computed from initial damping ratio (ATC 40 suggests an initial elastic viscous damping ratio of 0.05 for reinforced concrete building) and the displacement ductility ratio (μ). ATC 40 provides the following equations to calculate equivalent time period (Teq) and equivalent damping (βeq).

$$T_{eq} = T_i\sqrt{\frac{\mu}{1+\alpha\mu-\alpha}}$$

$$\beta_{eq} = \beta_i + \kappa\frac{2}{\pi}\frac{(\mu-1)(1-\alpha)}{\mu(1+\alpha\mu-\alpha)} = 0.05 + \kappa\frac{2}{\pi}\frac{(\mu-1)(1-\alpha)}{\mu(1+\alpha\mu-\alpha)}$$

where α is the post-yield stiffness ratio and κ is an adjustment factor to approximately account for changes in hysteretic behaviour in reinforced concrete structures.

The equivalent period in equation is based on a lateral stiffness of the equivalent system that is equal to the secant stiffness at the target displacement. This equation does not depend on the degrading characteristics of the hysteretic behaviour of the system. It only depends on the displacement ductility ratio (μ) and the post-yield stiffness ratio (α) of the inelastic system.

Since the equivalent period and equivalent damping are both functions of the displacement ductility ratio (Equations), it is required to have prior knowledge of displacement ductility ratio. However, this is not known at the time of evaluating a structure. Therefore, iteration is required to determine target displacement. ATC 40 describes three iterative procedures with different merits and demerits to reach the solution.

2.15 Hysteretic model

The hysteretic model incorporates

- stiffness degradation (α), strength deterioration (β),
- pinching behavior (γ). The hysteretic model for flexural
- response is based on Takeda model (Kunnath *et al*., 1990).
- The Takeda hysteresis model was developed by Takeda, Sozen and Nielsen [1970], Otani [1981] and Kabeyasawa, Shiohara, Otani, Aoyama [1983] to represent the force-displacement hysteretic properties of RC structures. The Takeda model according to Otani (1981) includes (a) stiffness changes at flexural Cracking and yielding, (b) rules for inner hysteresis loops inside the outer loop, and (c) unloading stiffness degradation with deformation. The hysteresis rules are extensive and comprehensive (Figure 1.1). In this work the modified Takeda Model [Ref: Kabeyasawa, Shiohara, Otani, Aoyama; May 1983. Analysis of the full-scale Seven storey Reinforced Concrete Test structure] is considered, in which the initial elastic branch up until cracking is neglected. Instead the response is linear up until yield with the unloading stiffness defined as

$$K_{un}=\frac{F_y}{D_y}*\left|\frac{D_m}{D_y}\right| \wedge -a$$

in which (*Dy*, *Fy*): yielding point deformation and resistance, D*m*: maximum deformation amplitude greater than Dy, *α*: unloading stiffness degradation parameter (normally between 0.0 and 0.6).

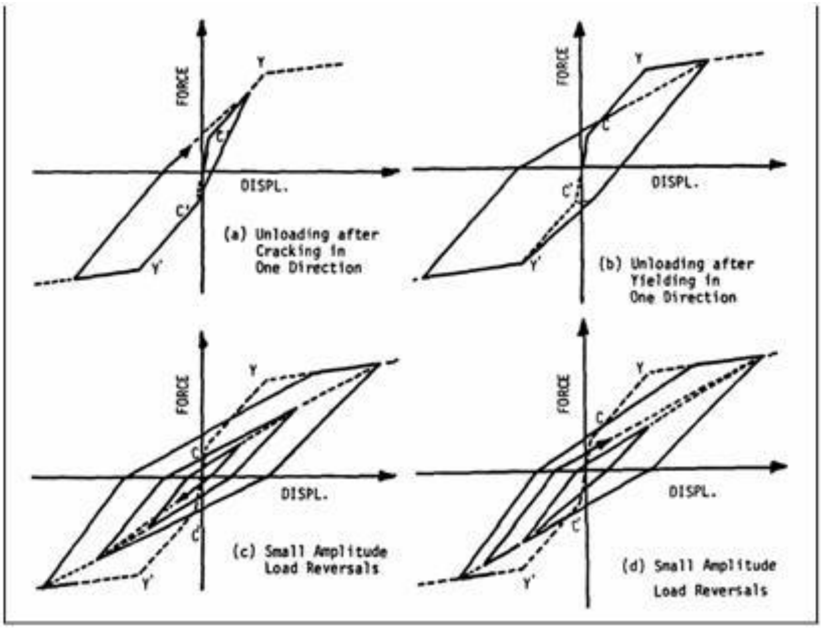


Figure No.:2.11.Takeda hysteresis model – Ref: Hysteresis Models of Reinforced Concrete for Earthquake Response Analysis by Otani [May 1981]

2.17 LITERATURE REVIEW

2.17.1 S. TALEBI1 AND M. R. KIANOUSH 2, BEHAVIOR OF REINFORCED CONCRETE FRAMES DESIGNED FOR DIFFERENT LEVELS OF DUCTILITY, 3th World Conference on Earthquake Engineering Vancouver, B.C., and Canada August 1-6, 2004 Paper No. 505-

According to the authors in this paper they described the seismic performance of 5 storey R.C.C frame designed and detailed according to Canadian practice. Analytical investigations were done using push over analysis. They concluded that ductile frame performed very well under push over analysis. For nominally ductile frame showed low ductility capacity even when they were stronger due to large member sizes.It resulted in single storey failure.

2.17.2 N.Choopool and V. Boonyapinyo , Seismic performance evaluation of reinforced concrete moment resisting frames with various ductility in low seismic zone.--

In this study they analysed the seismic performance and new cost estimates of nine storey building in Bangkok and compared the result with gravity load design. They assumed R= 8,5,3 for special ductile frame, intermediate frame and ordinary ductile frame respectively. It was concluded that SDF is more ductile than IDF and ODF but strength of ODF was more than SDF. ODF was most expensive of all three frames. The frames were designed with the objective of achieving immediate occupancy performance level.The main conclusion was that SDF and IDF was the best option for seismic performance and economy wise also.

2.17.3 Prashant Sunagar and S.M Shivananda, Evaluation of seismic response modification factors for R.C.C frames by Non Linear Analysis, Proceedings of International conference on Advances in Architecture and Civil Engineering (AARCV 2012), 21- 23rd June 2012.—

In this study 3,9,20 stories R.C.C moment resisting frames were taken.The lateral load carrying capacity and seismic response modification factor of individual frames were analysed. The frames were designed according to IS 456-2000 and IS 1893-2002, and provisions of FEMA. In this response spectrum and push over analysis is done It has paid great emphasis on R value.In equal displacement concept post elastic behaviour of the structure is neglected.Equal energy concept takes into account some concepts. These approaches are unrealistic.R values has not taken into consideration the strength issues.According to this thesis incorporating the other parameters into R will give more reliable results.This study reveals that current Indian code has not thrown any light on redundancy of structures.

2.17.4 Iona Olteanu, Ioan-Petru, Ciongradi,Mihaela Anechitei and M.Budescu, The ductile design concept for seismic actions in miscellaneous design codes—

In this paper the authors has presented the ductility concepts by comparing different international codes and has also compared the R value.

2.17.5 Naeim et. al. (2001) described the seismic performance of buildings and performance objectives to define the state of the building following a design earthquake. They also outlined the promises and limitations of performance based seismic engineering. They introduced and discussed the methodologies and techniques embodied in the two leading guidelines of this subject i.e. ATC- 40 and FEMA-273/274. They provided some numerical examples to illustrate the practical applications of the methods used.

CHAPTER 3

STRUCTURAL MODELLING

3.1 INTRODUCTION

The study in this thesis is based on nonlinear analysis of a family of structural models representing different material properties of concrete and steel. The first part of this chapter presents a summary of various parameters defining the computational models, the basic assumptions and the building geometries considered for this study. Accurate modelling of the nonlinear properties of various structural elements is very important in nonlinear analysis. The second part of this chapter presents the properties of the plastic hinges, the procedure to generate these hinge properties and the assumptions made. Finally, this chapter presents the important parameters used for pushover analysis.

3.2 COMPUTATIONAL MODEL

Modelling a building involves the modelling and assemblage of its various load-carrying elements. The model must ideally represent the mass distribution, strength, stiffness and deformability. Modelling of the material properties and structural elements used in the present study is discussed below.

3.2.1 Material Properties

Table 3.1: Assumed frames with material properties

FRAME	GRADE OF CONCRETE	REBAR MATERIAL
1	**M30**	**Fe415**
2	**M40**	**Fe415**
3	**M30**	**Fe500**
4	**M40**	**Fe500**

Elastic material properties of these materials are taken as per Indian Standard IS 456 (2000). The short-term modulus of elasticity (Ec) of concrete is taken as:

5000 $\sqrt{f_{ck}}$ MPa Where f_{ck} = characteristic compressive strength of concrete cube in MPa at 28-day (20 MPa in this case). For the steel rebar, yield stress (fy) and modulus of elasticity (Es) is taken as per IS 456 (2000).

3.2.2 The stress-strain curve of concrete

The stress-strain curve of concrete in compression forms the basis for analysis of any reinforced concrete section. The characteristic and design stress-strain curves specified in most of design codes (IS 456: 2000, BS 8110) do not truly reflect the actual stress-strain behaviour in the post-peak region, as (for convenience in calculations) it assumes a constant stress in this region (strains between 0.002 and 0.0035). In reality, as evidenced by experimental testing, the post-peak behaviour is characterised by a descending branch, which is attributed to ‘softening’ and micro-cracking in the concrete. Also, models as per these codes do not account for strength enhancement and ductility due to confinement. However, the stress-strain relation specified in ACI 318M-02 consider some of the important features from actual behaviour. A previous study (Chugh, 2004) on stressstrain relation of reinforced concrete section concludes that the model proposed by Panagiotakos and Fardis (2001) represents the actual behaviour best for normal-strength concrete. Accordingly, this model has been selected in the present study for calculating the hinge properties. This model is a modified version of Mander's model (Mander et. al., 1988) where a single equation can generate the stress fc corresponding to any given strain εc:

$$f_c = \frac{f'_{cc}\, x\, r}{r - 1 + x^r}$$

where, $x = \dfrac{\varepsilon_c}{\varepsilon_{cc}}$; $r = \dfrac{E_c}{E_c - E_{sec}}$; $E_c = 5000\sqrt{f'_{co}}$; $E_{sec} = \dfrac{f'_{cc}}{\varepsilon_{cc}}$; and f'_{cc} is the peak strength

expressed as follows:

$$f'_{cc} = f'_{co}\left[1 + 3.7\left(\frac{0.5 k_e \rho_s f_{yh}}{f'_{co}}\right)^{0.85}\right]$$

The expressions for critical compressive strains are expressed in this model as follows:

$$\varepsilon_{cu} = 0.004 + \frac{0.6 \rho_s f_{yh} \varepsilon_{sm}}{f'_{cc}}$$

$$\varepsilon_{cc} = \varepsilon_{co}\left[1 + 5\left(\frac{f'_{cc}}{f'_{co}} - 1\right)\right]$$

where, f_{co} is unconfined compressive strength = 0.75 fck, ρs = volumetric ratio of confining steel, f_{yh} = grade of the stirrup reinforcement, εsm = steel strain at maximum tensile stress and k_e is the "confinement effectiveness coefficient", having a typical value of 0.95 for circular sections and 0.75 for rectangular sections.

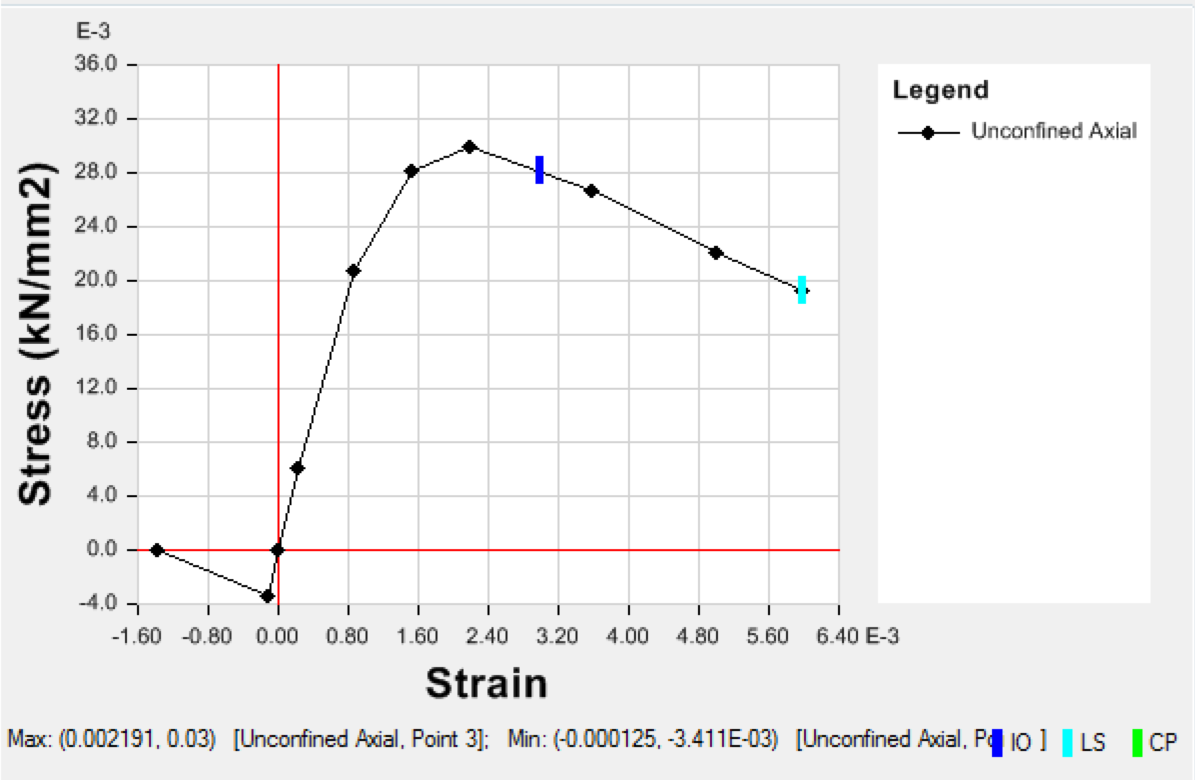


Figure No. 3.1 Mander's stress-strain graph for M30 unconfined axial concrete

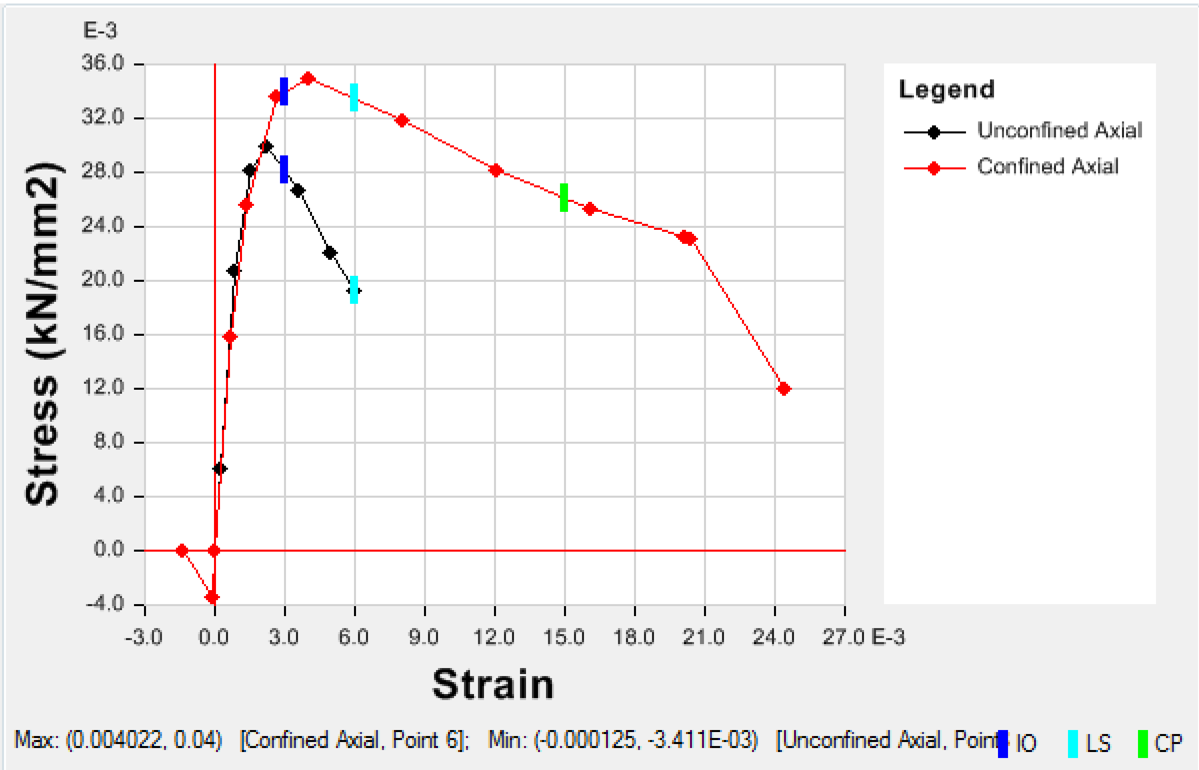


Figure No. 3.2 Stress – strain curve for unconfined and confined axial M30 concrete for column size 450mm x 500mm

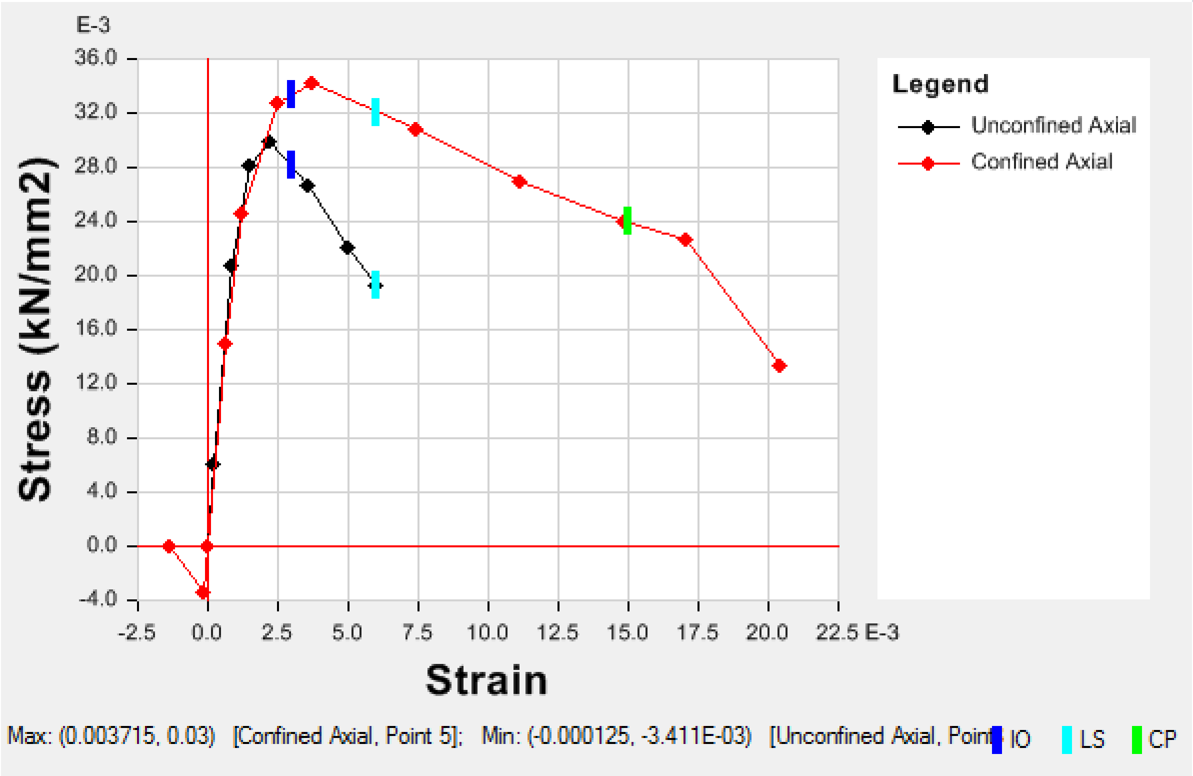


Figure No. 3.3 Stress- strain curves for confined and unconfined uniaxial M30 concrete for column size 600mm x 600 mm

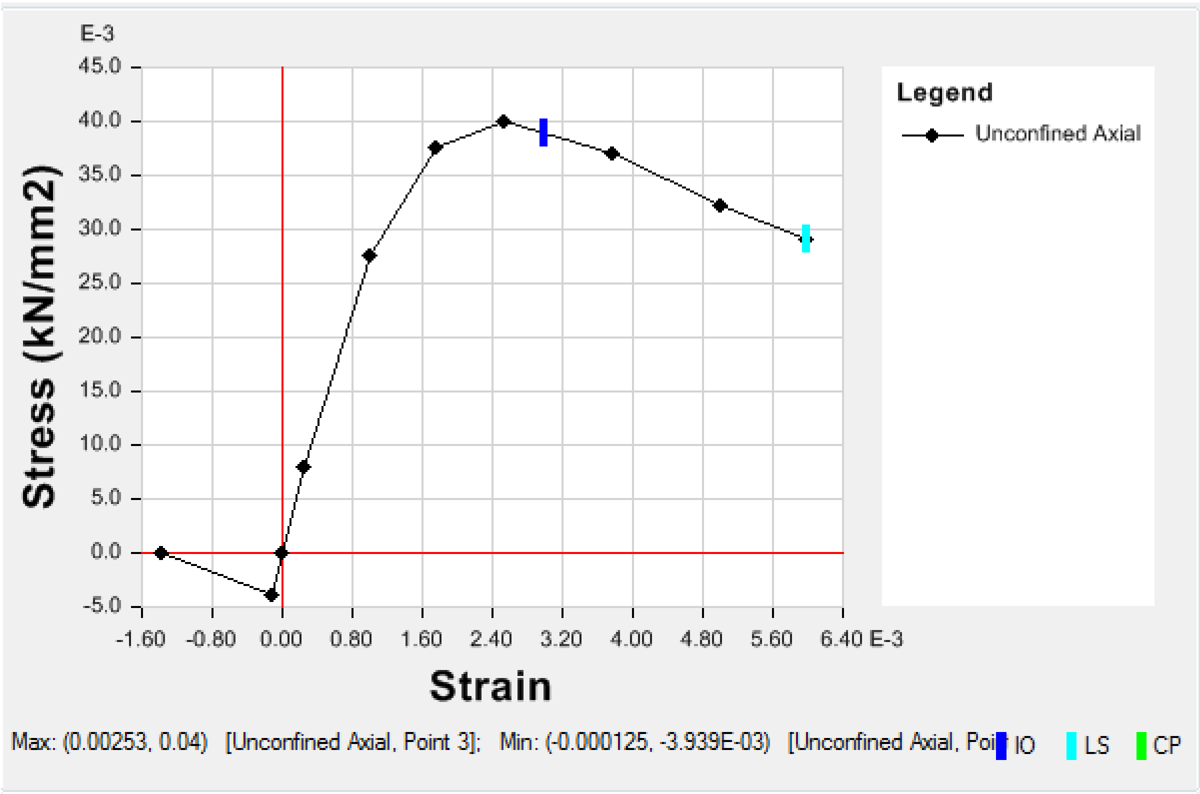


Figure No.3.4 Stress- strain curves for confined and unconfined uniaxial M40 concrete

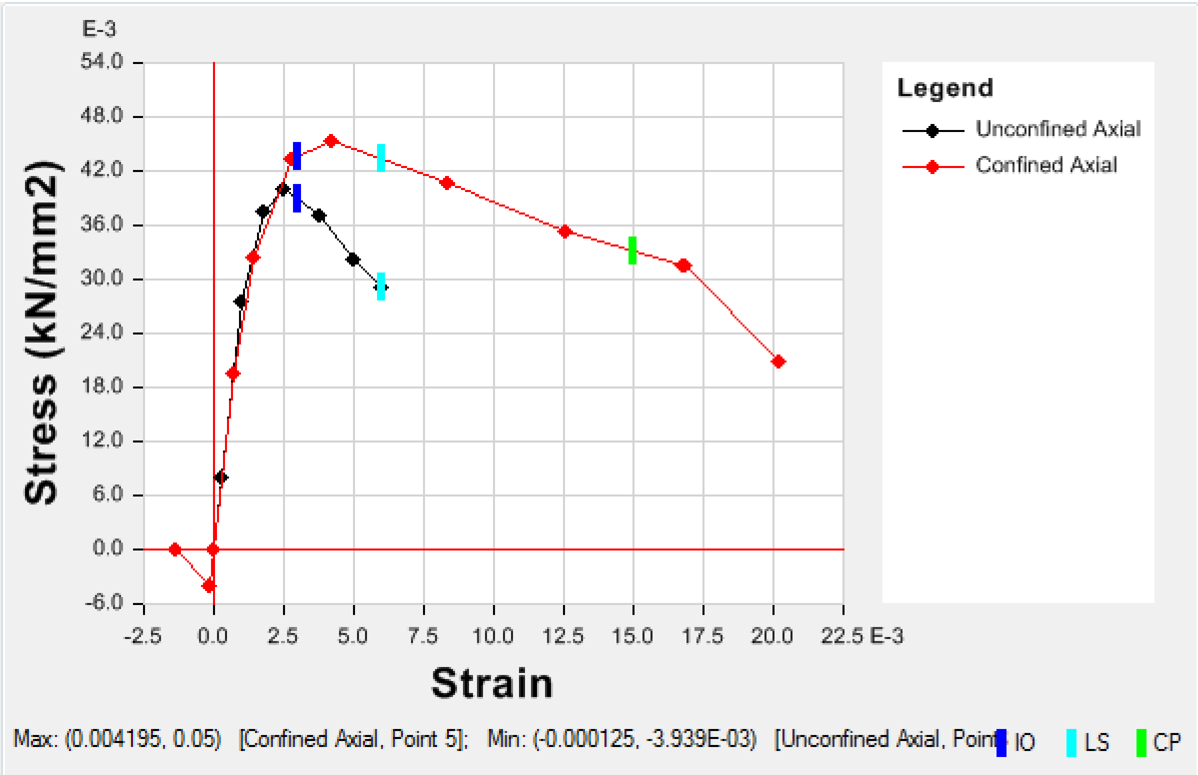


Figure No.3.5 Stress- strain curves for confined and unconfined axial M40 concrete for column size 450mm x 500 mm

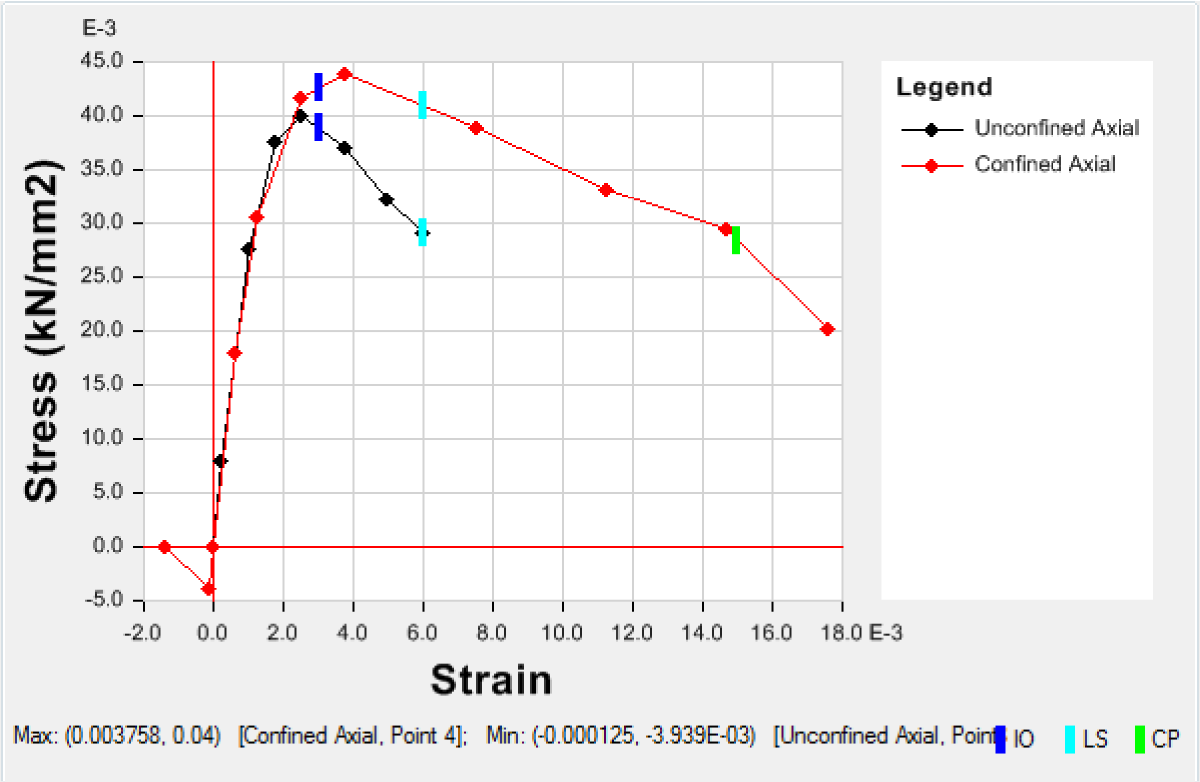


Figure No.3.6 Stress- strain curves for confined and unconfined axial M40 concrete for column size 450mm x 500 mm

3.2.3 Stress-Strain Characteristics for Reinforcing Steel

The 'characteristic' and 'design' stress-strain curves specified by the Code for Fe-415 grade of reinforcing steel (in tension or compression) are shown in Fig

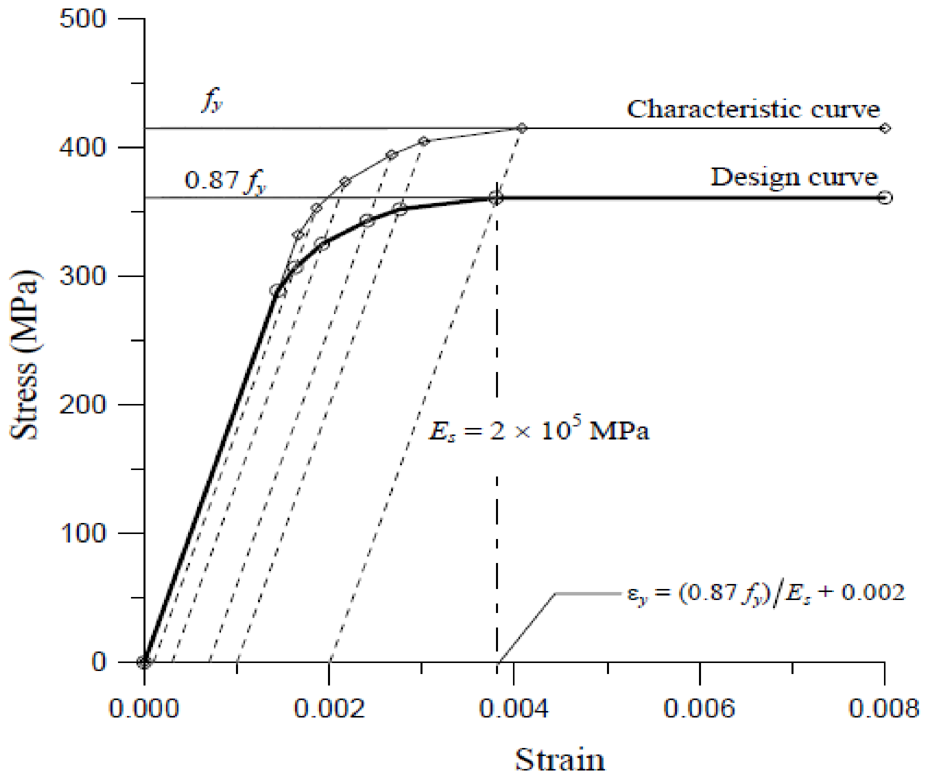


Figure No.3.7: Stress-strain relationship for reinforcement – IS 456 (2000)

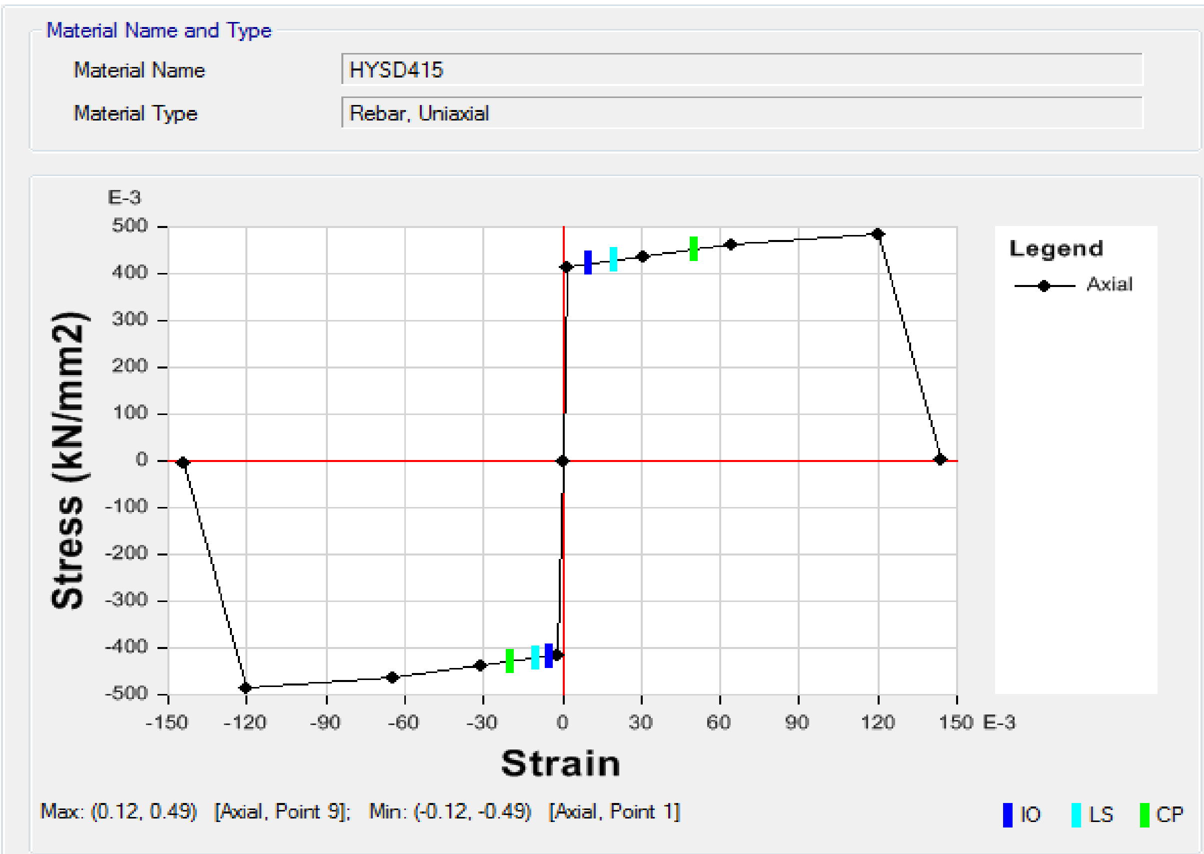


Figure No.3.8: Stress-strain relationship for HYSD 415 bars

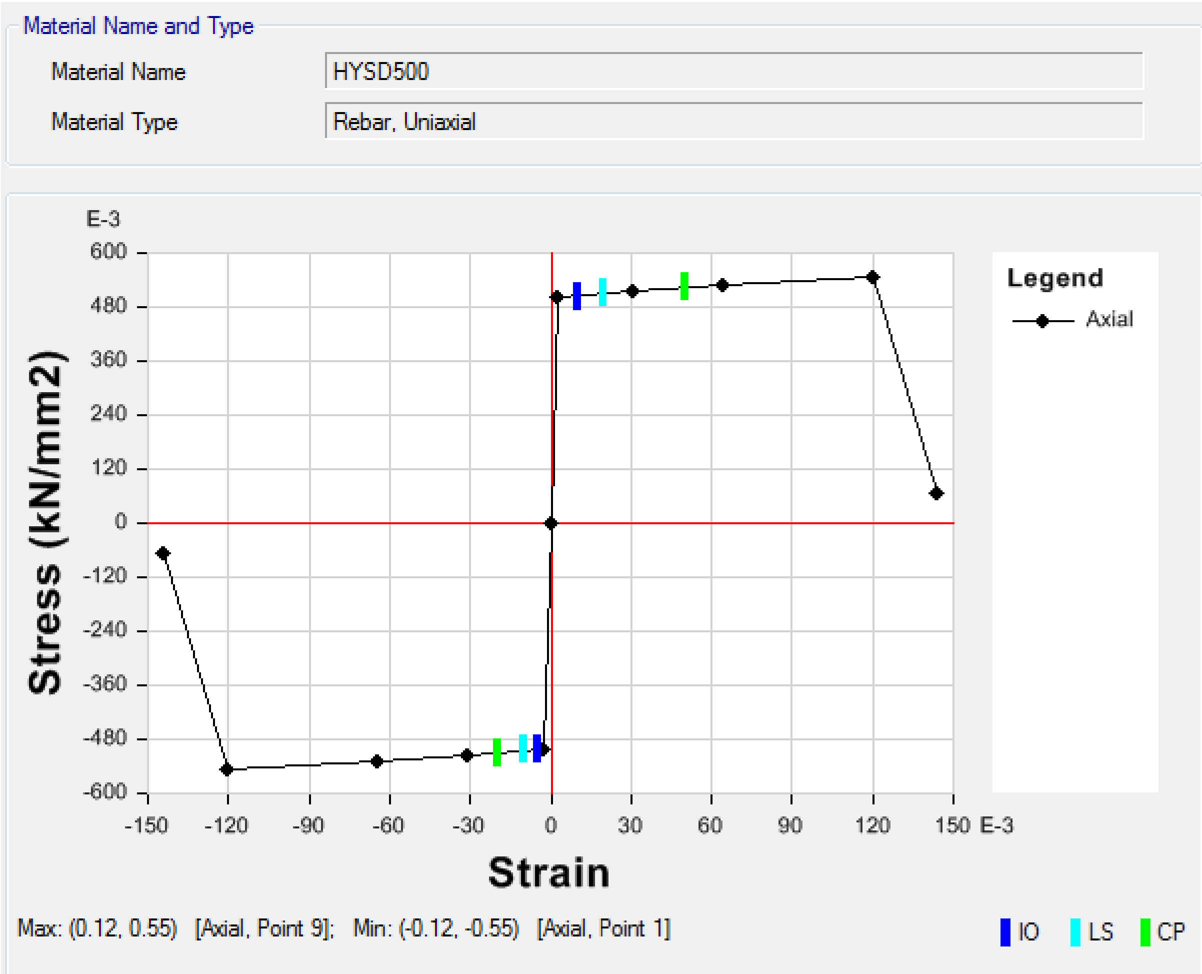


Figure No.3.9: Stress-strain relationship for Fe 500 bars

3.2.4 Sectional properties

For all frames used in the study following sectional properties are used

1. Beams (450x600) mm
2. Column (600x600) mm
3. Column (450x500) mm

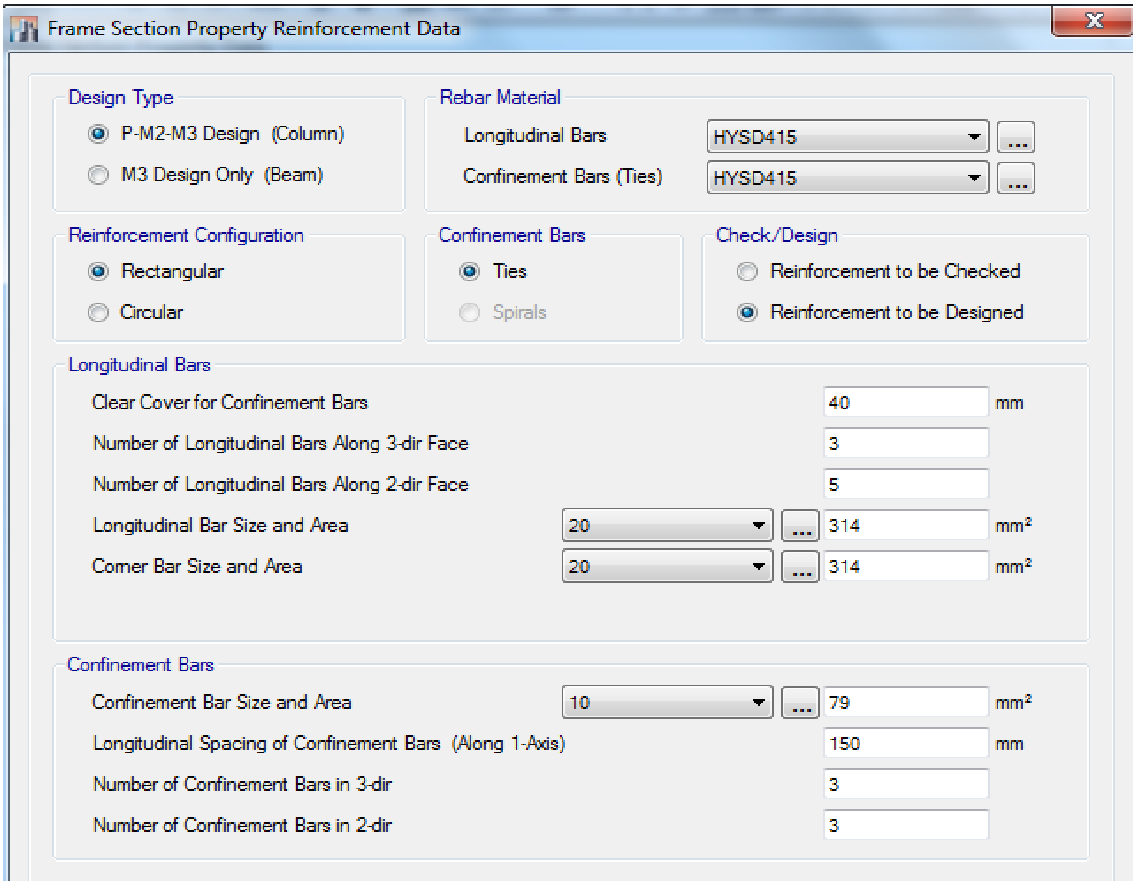


Figure No.3.10 Frame section property reinforcement data for column size 600mm x 600mm

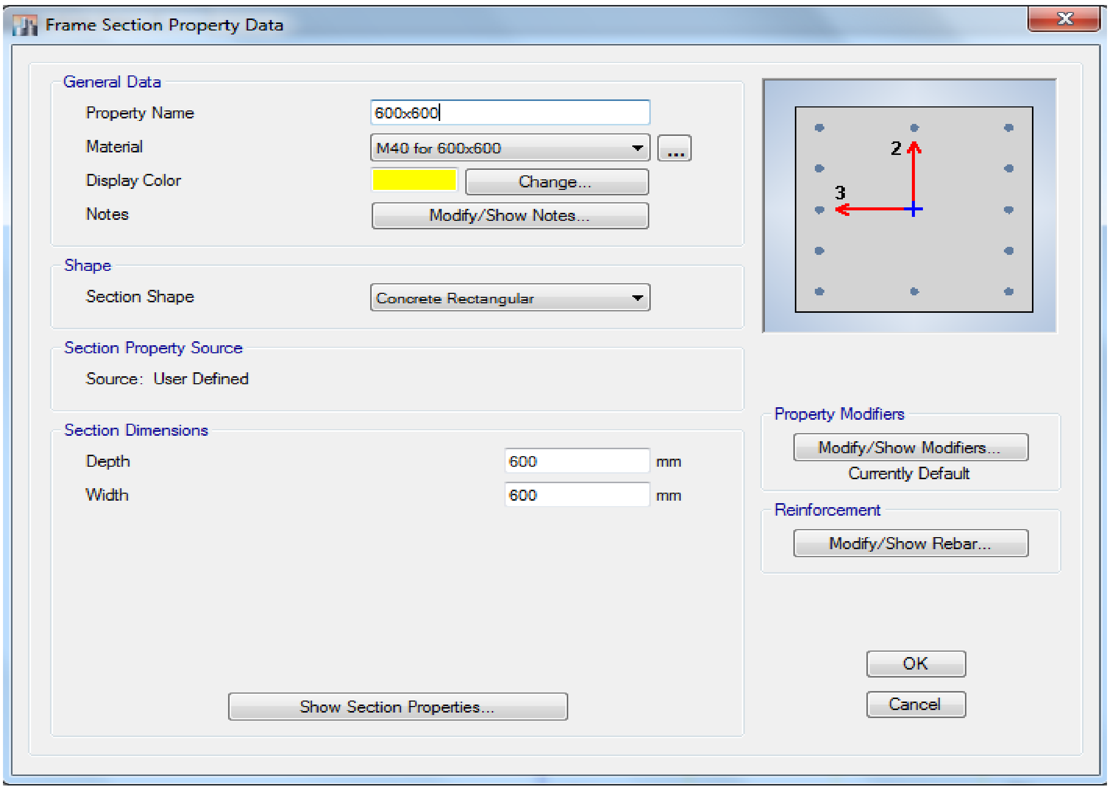


Figure No.3.11 Frame section property data for column 600mm x 600mm

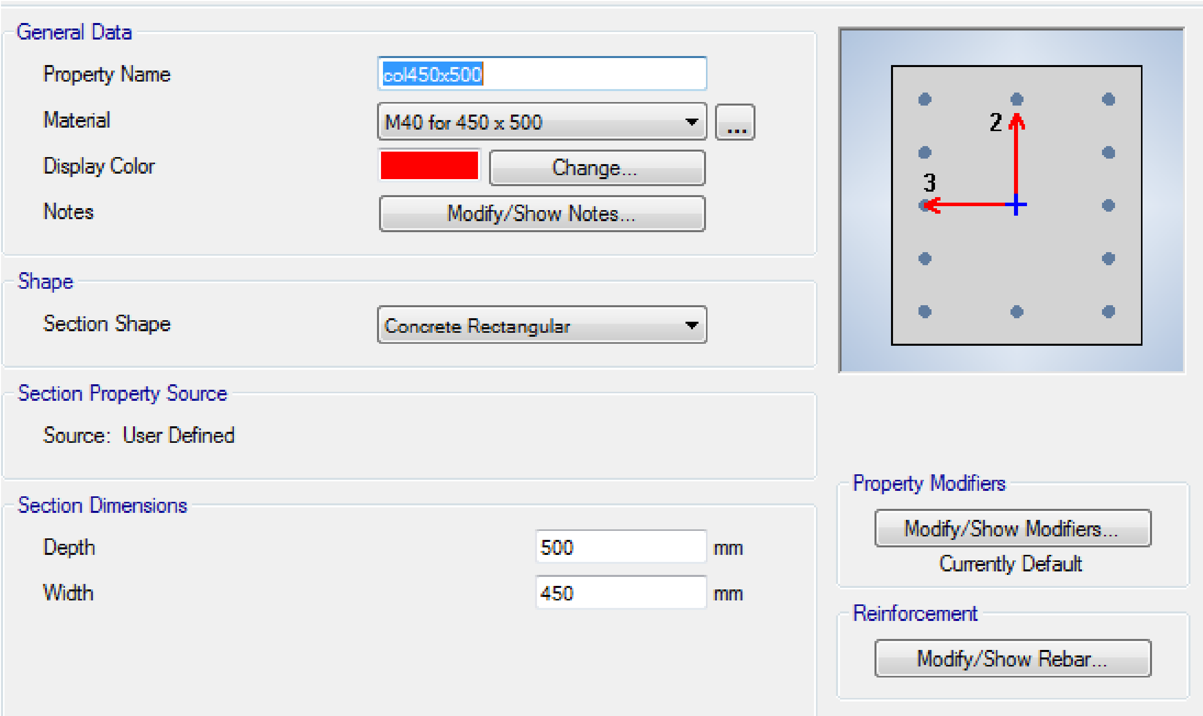


Figure No.3.12 Frame section property data for column 450mm x 500mm

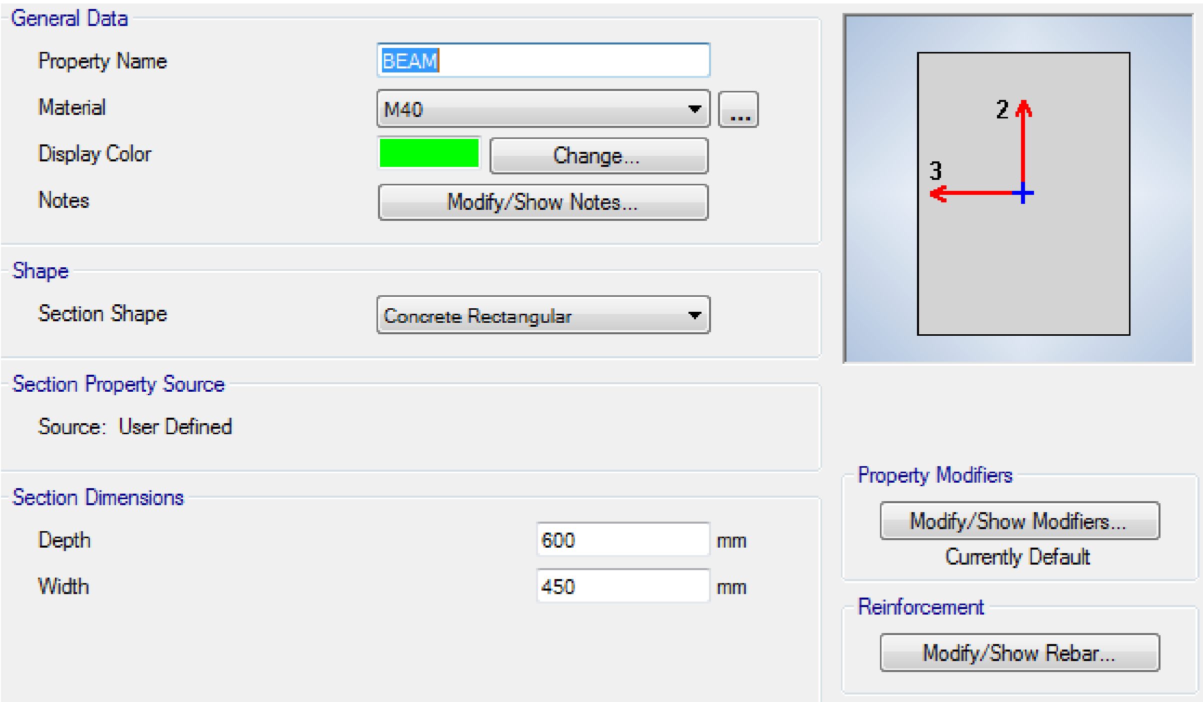


Figure No.3.13 Frame section property data for beam

3.2.5 Supports and Restraints

The column end at foundation was considered as fixed for all the models in this study. All the frame elements are modelled with nonlinear properties at the possible yield locations.

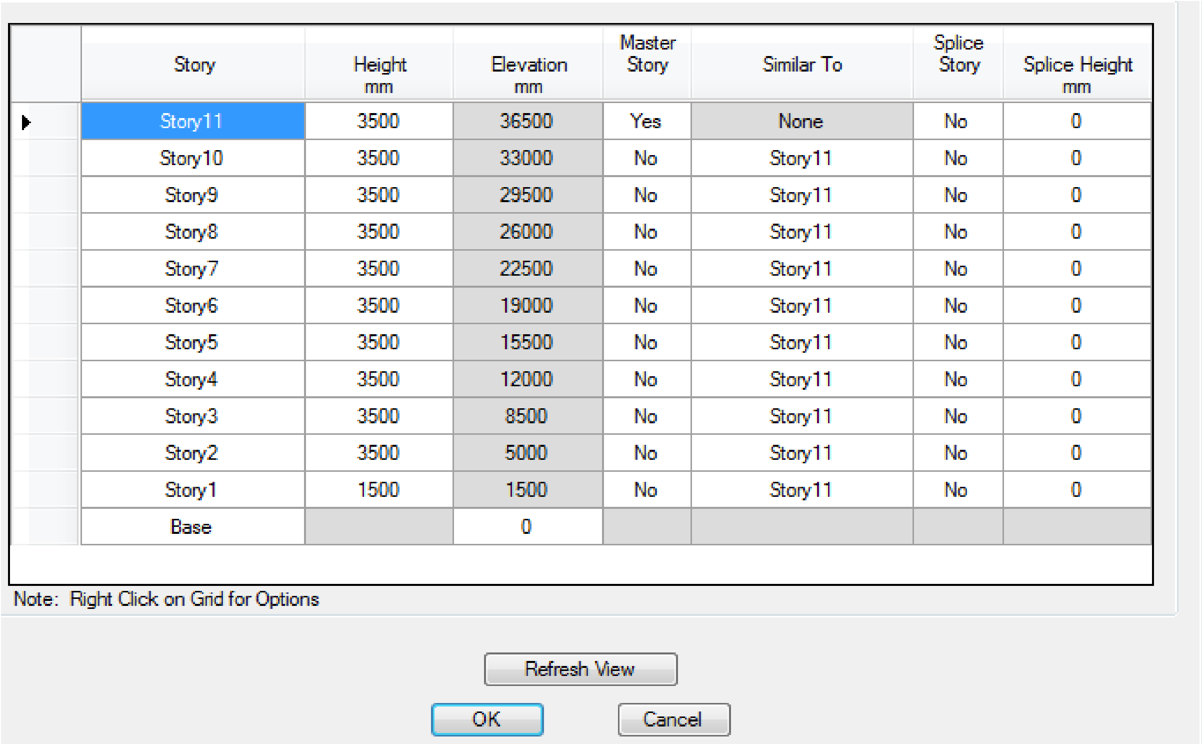
Figure No.3.14 Frame elevation showing supports

3.3 BUILDING GEOMETRY

The study is based on frames which are plane and orthogonal with storey heights and bay widths.

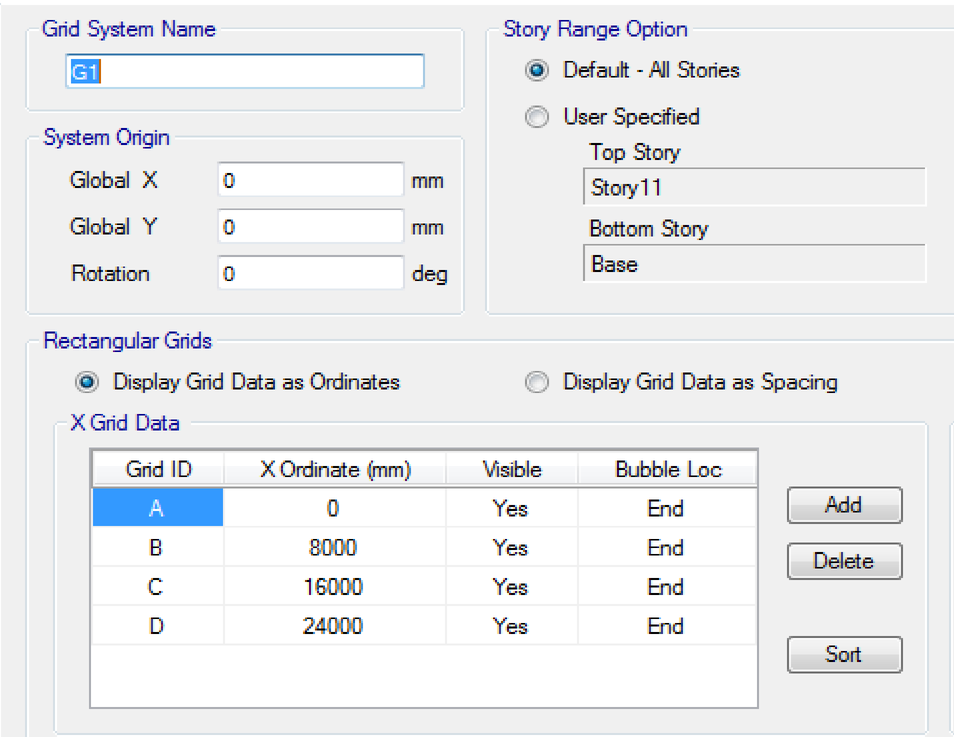
Table 3.2 Storey geometry

Name	Height mm	Elevation mm	Master Story	Similar To	Splice Story
Story11	3500	36500	Yes	None	No
Story10	3500	33000	No	Story11	No
Story9	3500	29500	No	Story11	No
Story8	3500	26000	No	Story11	No
Story7	3500	22500	No	Story11	No
Story6	3500	19000	No	Story11	No
Story5	3500	15500	No	Story11	No
Story4	3500	12000	No	Story11	No
Story3	3500	8500	No	Story11	No
Story2	3500	5000	No	Story11	No
Story1	1500	1500	No	Story11	No
Base	0	0	No	None	No



	Story	Height mm	Elevation mm	Master Story	Similar To	Splice Story	Splice Height mm
▶	Story11	3500	36500	Yes	None	No	0
	Story10	3500	33000	No	Story11	No	0
	Story9	3500	29500	No	Story11	No	0
	Story8	3500	26000	No	Story11	No	0
	Story7	3500	22500	No	Story11	No	0
	Story6	3500	19000	No	Story11	No	0
	Story5	3500	15500	No	Story11	No	0
	Story4	3500	12000	No	Story11	No	0
	Story3	3500	8500	No	Story11	No	0
	Story2	3500	5000	No	Story11	No	0
	Story1	1500	1500	No	Story11	No	0
	Base		0				

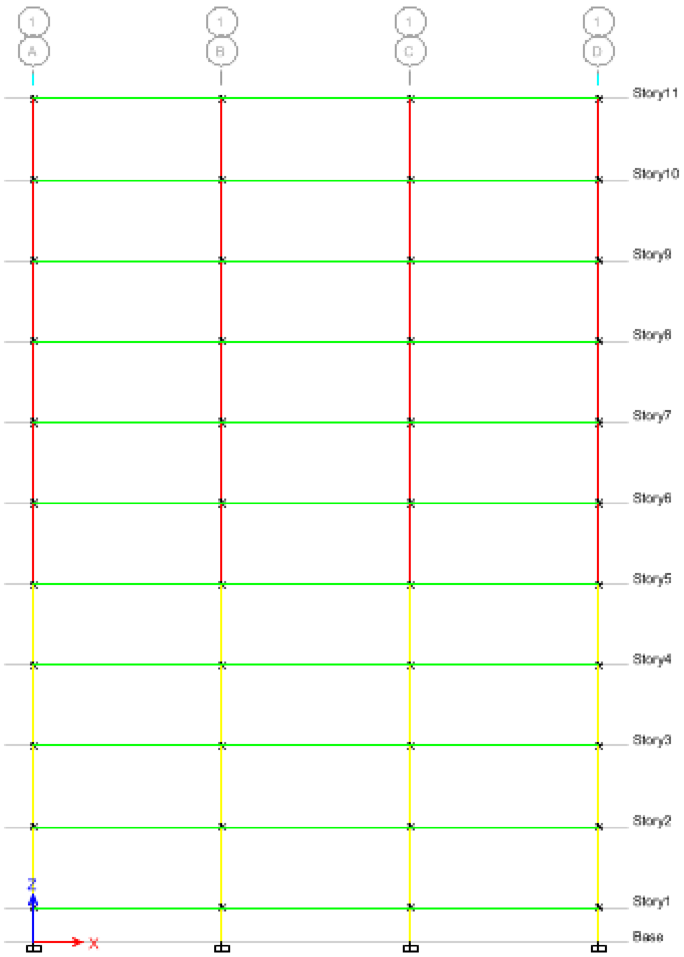
Figure No.3.15 Storey data



Grid ID	X Ordinate (mm)	Visible	Bubble Loc
A	0	Yes	End
B	8000	Yes	End
C	16000	Yes	End
D	24000	Yes	End

Figure No.3.16 Grid data

Figure No.3.17 Plan of First storey



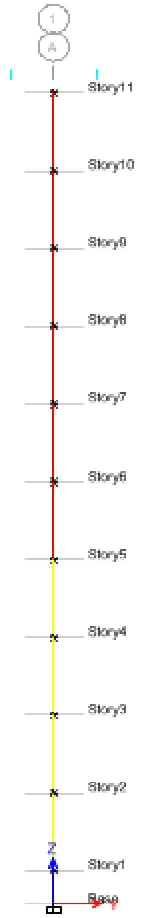


Figure No. 3.18 Elevation of Frame

Figure No. 3.19 Sectional Elevation of Frame

3.4 LOADS ASSIGNMENTS

Table 3.3 Load cases

Name	Type	Self Weight Multiplier	Auto Load
Dead	Dead	1	
Live	Live	0	
EQx	Seismic	0	None
FF+slab	Superimposed Dead	0	

Table 3.4 Load patterns

Story	Label	Unique Name	Load Pattern	FX kN	FY kN	FZ kN	MX kN-mm	MY kN-mm	MZ kN-mm
Story11	1	41	EQx	100	0	0	0	0	0
Story10	1	37	EQx	100	0	0	0	0	0
Story9	1	33	EQx	100	0	0	0	0	0
Story8	1	29	EQx	100	0	0	0	0	0
Story7	1	25	EQx	100	0	0	0	0	0
Story6	1	21	EQx	100	0	0	0	0	0
Story5	1	17	EQx	100	0	0	0	0	0
Story4	1	13	EQx	100	0	0	0	0	0
Story3	1	9	EQx	100	0	0	0	0	0
Story2	1	5	EQx	100	0	0	0	0	0

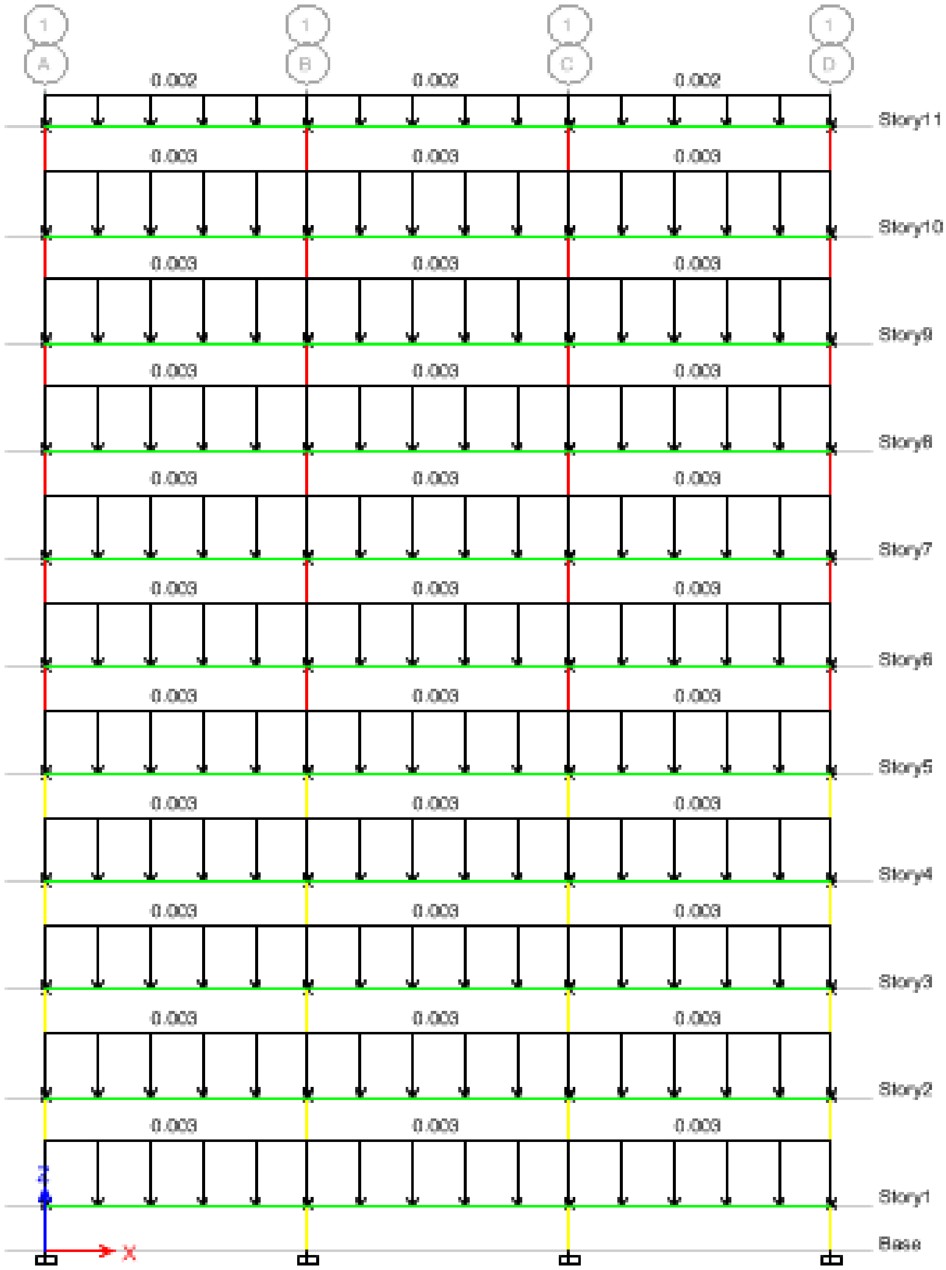


Figure No.3.20 Live Loads

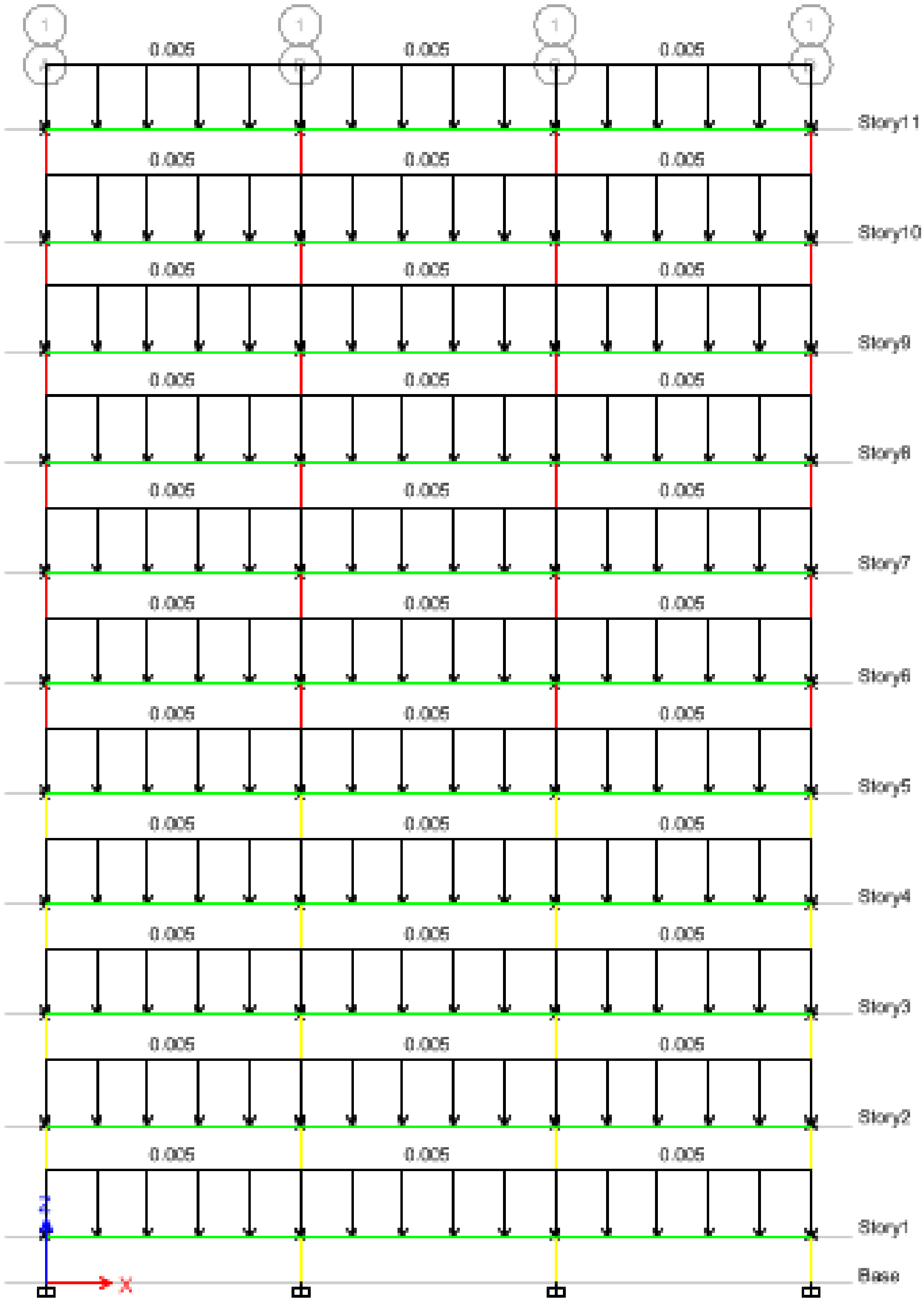


Figure No.3.21 Floor finish and Slab load

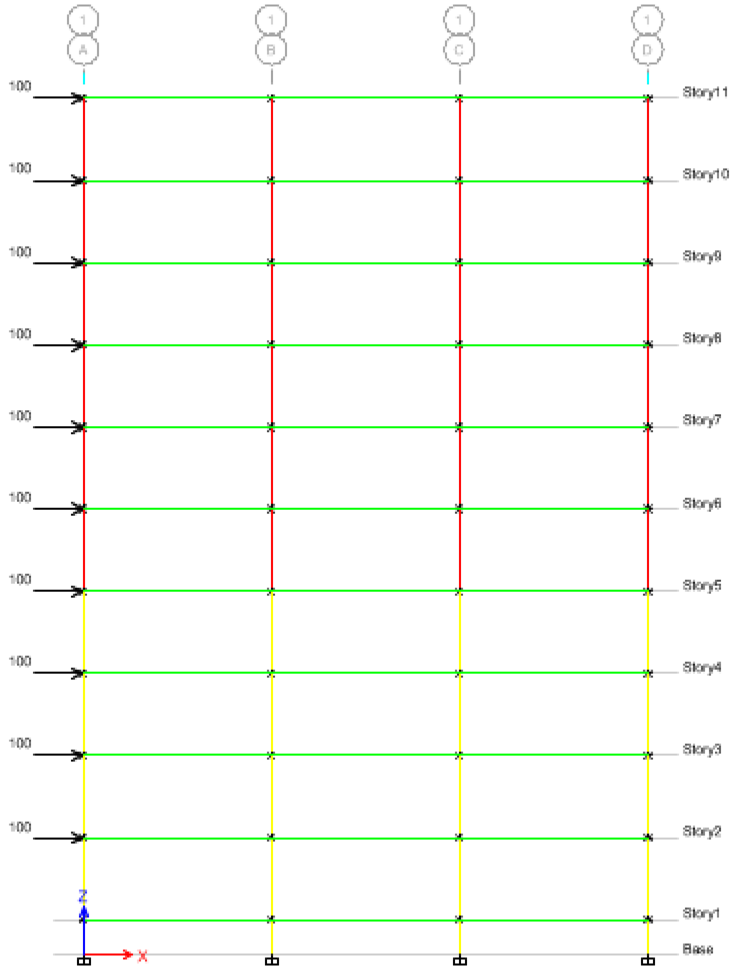


Figure No. 3.22 Lateral Earthquake load

Table 3.5 load cases summary

Name	Type
Dead	Linear Static
Live	Linear Static
EQx	Linear Static
FF+slab	Linear Static
Push dead	Nonlinear Static
pushx	Nonlinear Static

Table 3.6 Load combination summary

Name	Load Case/Combo	Scale Factor	Type	Auto
DCon1	Dead	1.5	Linear Add	Yes
DCon1	FF+slab	1.5		No
DCon2	Dead	1.5	Linear Add	Yes
DCon2	Live	1.5		No
DCon2	FF+slab	1.5		No
DCon3	Dead	1.2	Linear Add	Yes
DCon3	Live	1.2		No
DCon3	FF+slab	1.2		No
DCon3	EQx	1.2		No
DCon4	Dead	1.2	Linear Add	Yes
DCon4	Live	1.2		No
DCon4	FF+slab	1.2		No
DCon4	EQx	-1.2		No
DCon5	Dead	1.5	Linear Add	Yes
DCon5	FF+slab	1.5		No
DCon5	EQx	1.5		No
DCon6	Dead	1.5	Linear Add	Yes
DCon6	FF+slab	1.5		No
DCon6	EQx	-1.5		No
DCon7	Dead	0.9	Linear Add	Yes
DCon7	FF+slab	0.9		No
DCon7	EQx	1.5		No
DCon8	Dead	0.9	Linear Add	Yes
DCon8	FF+slab	0.9		No
DCon8	EQx	-1.5		No

3.5 MODELLING OF FLEXURAL PLASTIC HINGES

In the implementation of pushover analysis, the model must account for the nonlinear behaviour of the structural elements. Beam and column elements in this study were modelled with flexure (M3 for beams and P-M2-M3 for columns) hinges at possible plastic regions under lateral load (i.e., both ends of the beams and columns). Properties of flexure hinges must simulate the actual response of reinforced concrete components subjected to lateral load. In the present study the plastic hinge properties are calculated by ETABS.

Flexural hinges in this study are defined by moment-rotation curves calculated based on the cross-section and reinforcement details at the possible hinge locations. For calculating hinge properties it is required to carry out moment–curvature analysis of each element. Constitutive relations for concrete and reinforcing steel, plastic hinge length in structural element are required for this purpose. The flexural hinges in beams are modelled with uncoupled moment

(M3) hinges whereas for column elements the flexural hinges are modelled with coupled P-M2-M3 properties that include the interaction of axial force and bi-axial bending moments at the hinge location. Although the axial force interaction is considered for column flexural hinges the rotation values were considered only for axial force associated with gravity load

3.6 MOMENT CURVATURE RELATIONSHIP

3.6.1 Introduction

Moment-curvature relation is a basic tool in the calculation of deformations in flexural members. It has an important role to play in predicting the behaviour of reinforced concrete (RC) members under flexure. In nonlinear analysis, it is used to consider secondary effects and to model plastic hinge behaviour. Curvature (φ) is defined as the reciprocal of the radius of curvature (R) at any point along a curved line. When an initial straight beam segment is subject to a uniform bending moment throughout its length, it is expected to bend into a segment of a circle with a curvature φ that increases in some manner with increase in the applied moment (M). Curvature φ may be alternatively defined as the angle change in the slope of the elastic curve per unit length ($\phi = 1/R = d\theta/ds$) . At any section, using the 'plane sections remain plane' hypothesis under pure bending, the curvature can be computed as the ratio of the normal strain at any point across the depth to the distance measured from the neutral axis at that section.

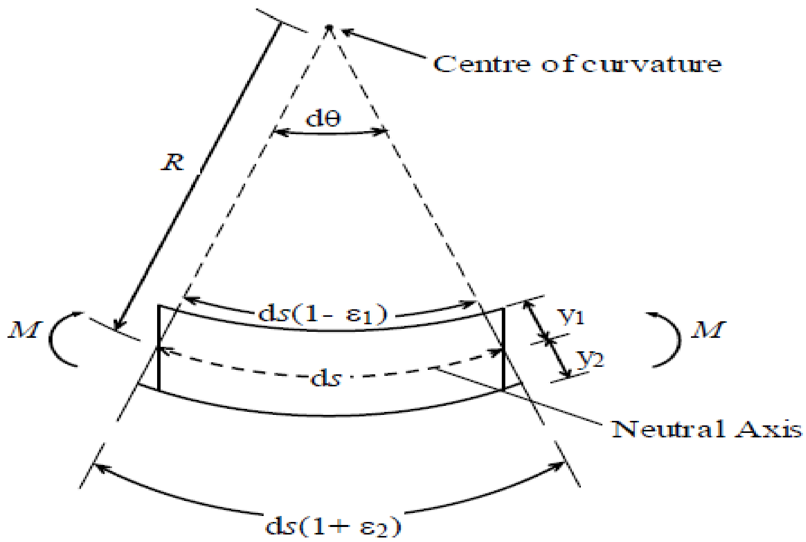


Figure No. 3.23: Curvature in an initially straight beam section

If the bending produces extreme fibre strains of ε_1 and ε_2 at top and bottom at any section as shown in Fig. 3.7 (compression on top and tension at bottom assumed in this case), then, for small deformations, it can be shown that $\phi = (\varepsilon_1 + \varepsilon_2)/D$, where D is the depth of the beam. If the beam behaviour is linear elastic, then the moment-curvature relationship is linear, and the curvature is obtained as

$$\Phi = \frac{M}{EI}$$

where EI is the flexural rigidity of the beam, obtained as a product of the modulus of elasticity E and the second moment of area of the section I.

When an RC flexural member is subjected to a gradually increasing moment, it's behaviour transits through various stages, starting from the initial un-cracked state to the ultimate limit state of collapse. The stresses in the tension steel and concrete go on increasing as the moment increases. The behaviour at the ultimate limit state depends on the percentage of steel provided, i.e., on whether the section is 'under-reinforced' or 'over-reinforced'. In the case of under-reinforced sections, failure is triggered by yielding of tension steel whereas in over-reinforced section the steel does not yield at the limit state of failure. In both cases, the failure eventually occurs due to crushing of concrete at the extreme compression fibre, when the ultimate strain in concrete reaches its limit. Under-reinforced beams are characterised by 'ductile' failure, accompanied by large deflections and significant flexural cracking. On the other hand, over-reinforced beams have practically no ductility, and the failure occurs suddenly, without the warning signs of wide cracking and large deflections.

3.6.2 Moment curvature in R.C sections

Using the Modified Mander model of stress-strain curves for concrete (Panagiotakos and Fardis, 2001) and Indian Standard IS 456 (2000) stress-strain curve for reinforcing steel, for a specific confining steel, moment curvature relations can be generated for beams and columns (for different axial load levels). The assumptions and procedure used in generating the moment-curvature curves are outlined below.

Assumptions

i. The strain is linear across the depth of the section ('plane sections remain plane').
ii. The tensile strength of the concrete is ignored.
iii. The concrete spalls off at a strain of 0.0035.
iv. The initial tangent modulus of the concrete, E_c is adopted from IS 456 (2000), as

$5000\sqrt{f_{ck}}$

v. In determining the location of the neutral axis, convergence is assumed to be reached within an acceptable tolerance of 1%.

3.6.3 Moment rotation for beams

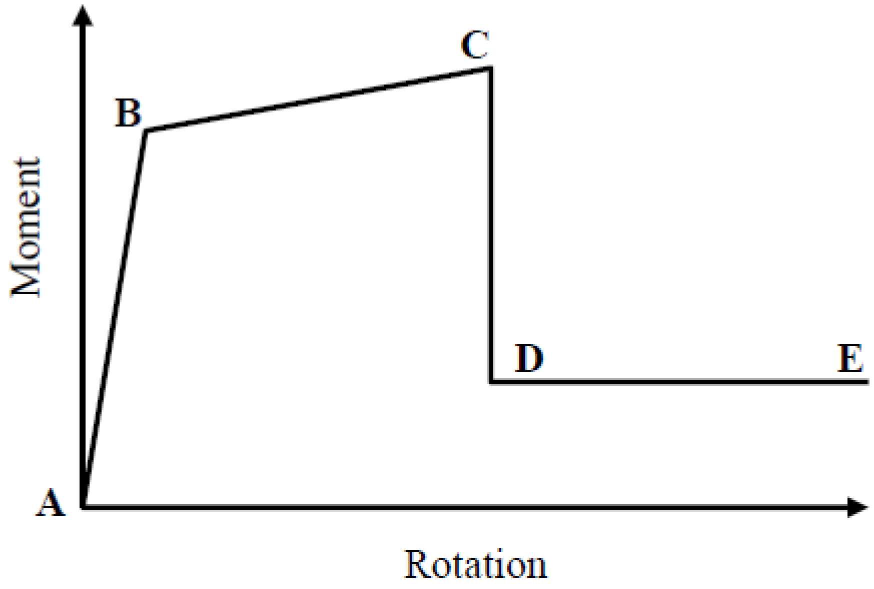


Figure No.3.24: Idealised moment-rotation curve of RC beam sections

・The point 'A' corresponds to the unloaded condition.

・The point 'B' corresponds to the nominal yield strength and yield rotation θy .

・The point 'C' corresponds to the ultimate strength and ultimate rotation θu , following which failure takes place.

・The point 'D' corresponds to the residual strength, if any, in the member. It is usually limited to 20% of the yield strength, and ultimate rotation, θu can be taken with that.

・The point 'E' defines the maximum deformation capacity and is taken as 15 θy or θu , whichever is greater.

3.6.4 Moment-Rotation Parameters for Columns (PMM Hinges)

For the PMM hinge, an interaction (yield) surface is specified in three-dimensional PM2-M3 space that represents where yielding first occurs for different combinations of axial force P, minor moment M2, and major moment M3. The surface is specified as a set of P-M2-M3 curves, where P is the axial force, and M2 and M3 are the moments

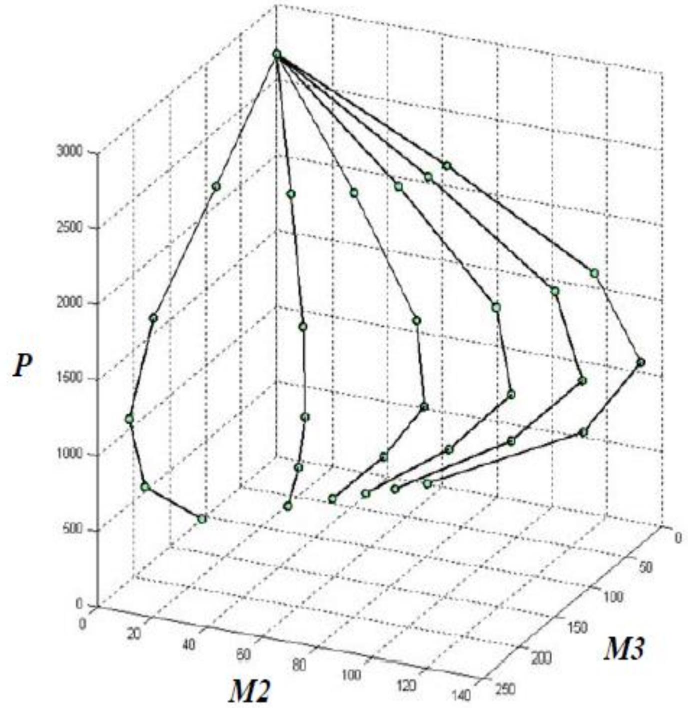


Figure No. 3.25: PMM Interaction Surface

For PMM hinges it requires to specify multiple moment -rotation curves corresponding to different values of P. Moment values normalized with yield moment. Yield moment is calculated from the PMM interaction surface for the appropriate axial force

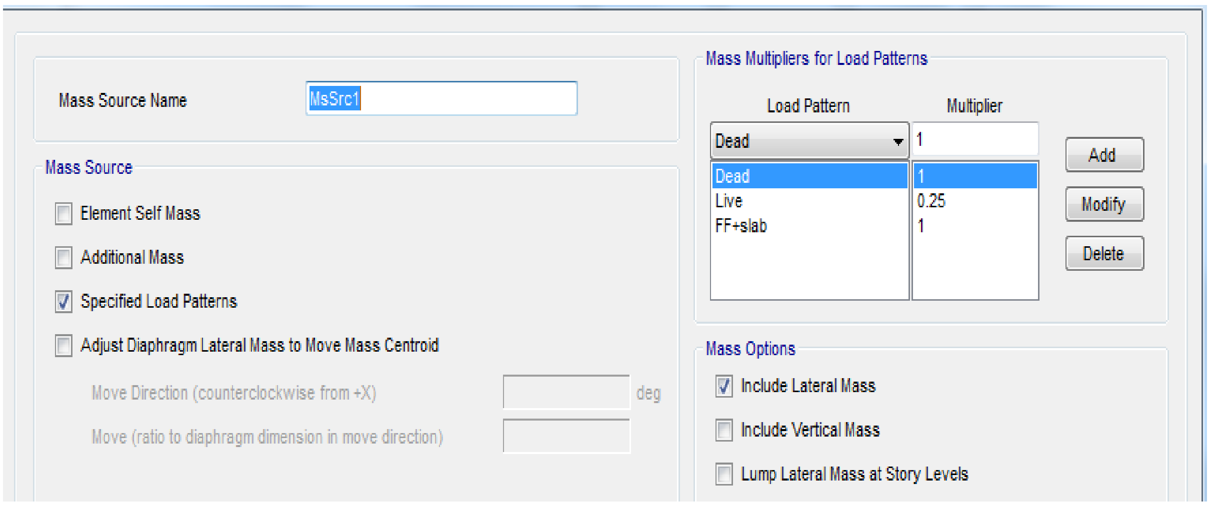


Figure No. 3.26 Mass source

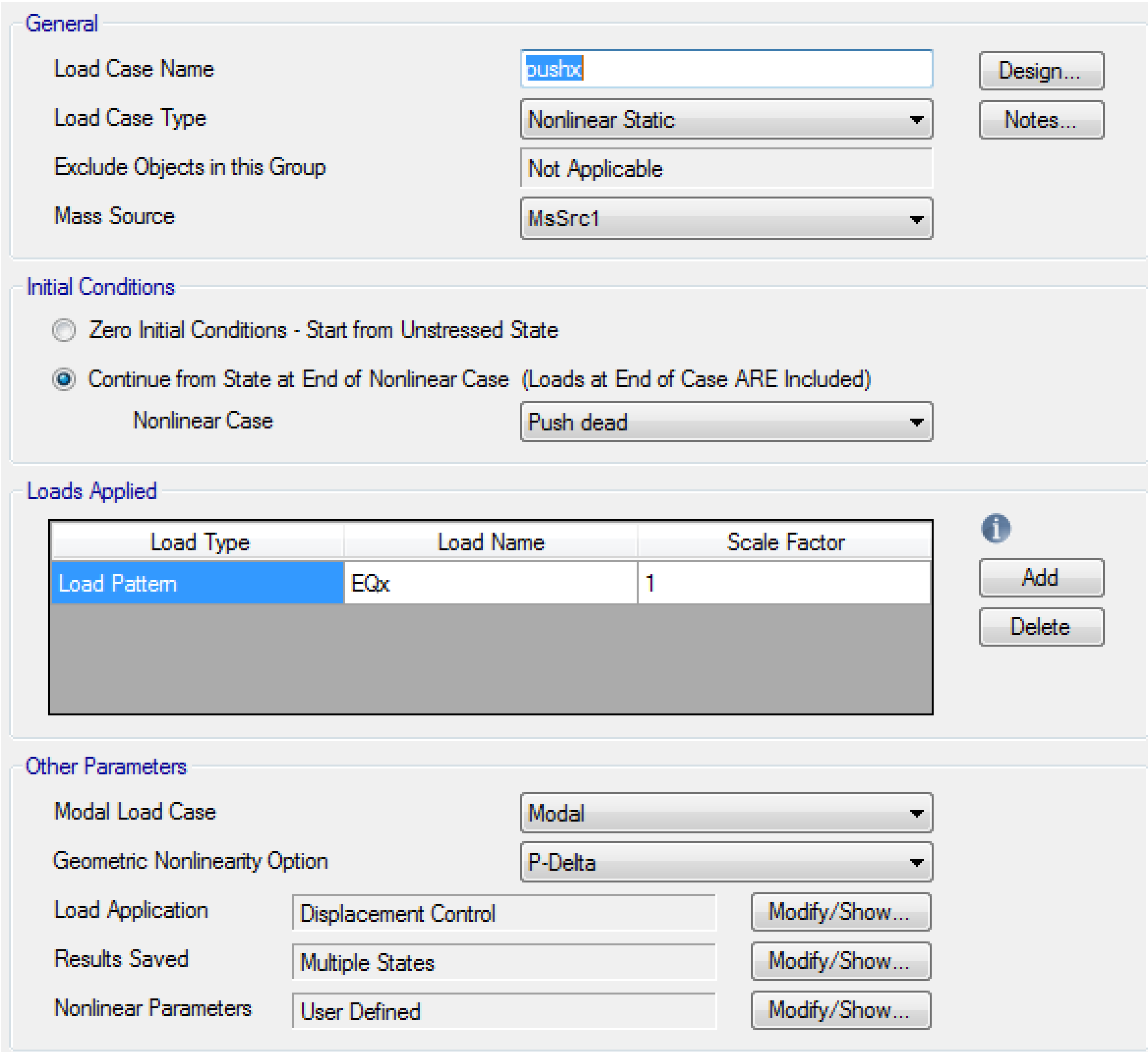


Figure No. 3.27 Data for Push x case

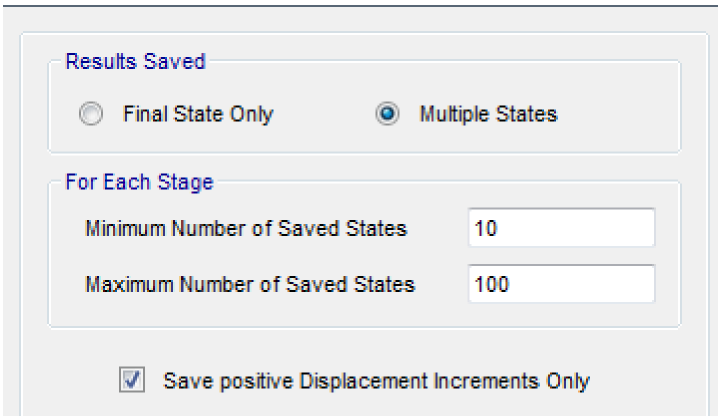


Figure No. 3.28 Data for Push x case

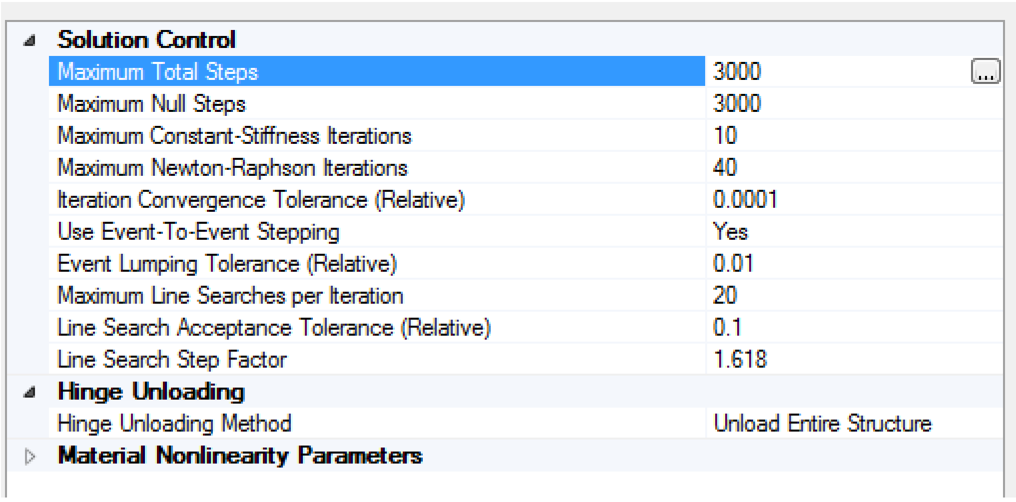


Figure No. 3.29 Data for Push x case

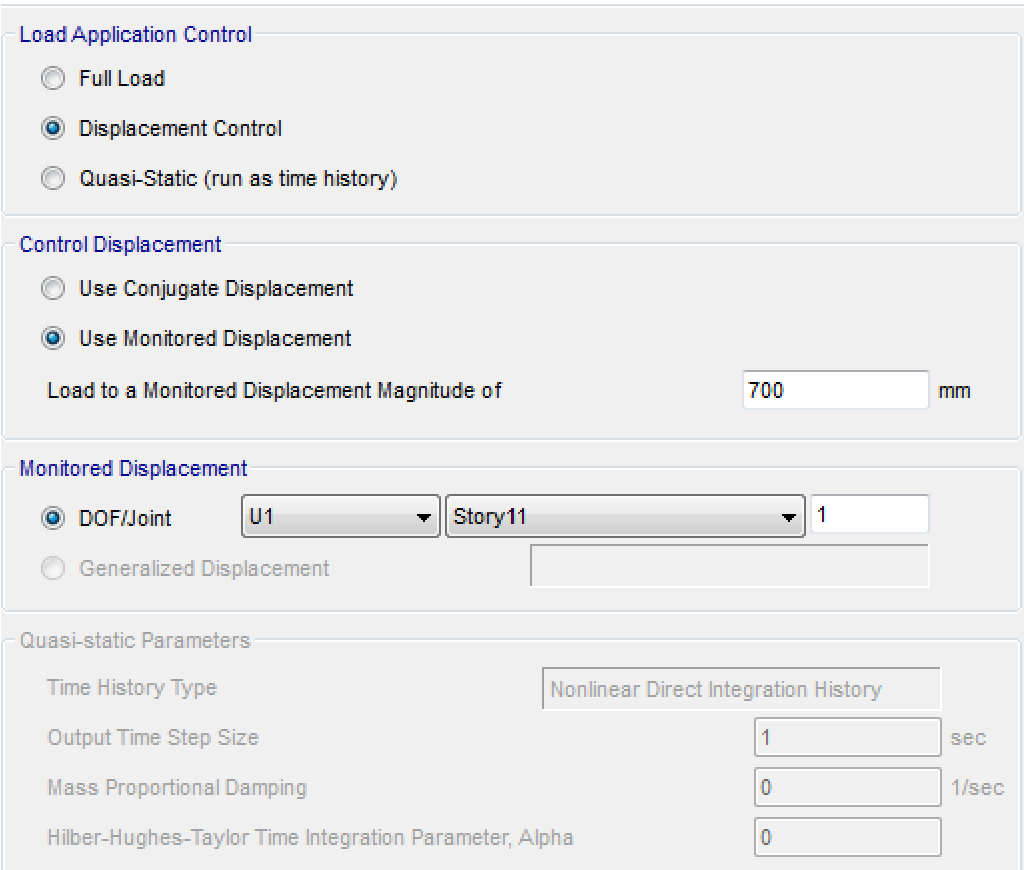


Figure No. 3.30 Data for Push x case

CHAPTER 4

ANALYSIS RESULTS

4.1 INTRODUCTION

All the 4 R.C frames with different properties presented in Chapter 3 are analyzed for linear/nonlinear static/dynamic behaviour using ETABS 2015. This chapter presents the results obtained from the above analyses. The results presented here are focussed on following two broad categories: (i) results from static and modal analysis (ii) Estimation of ductility.

4.2 STATIC ANALYSIS

4.2.1 FRAME 1 M30 grade concrete and Fe 415

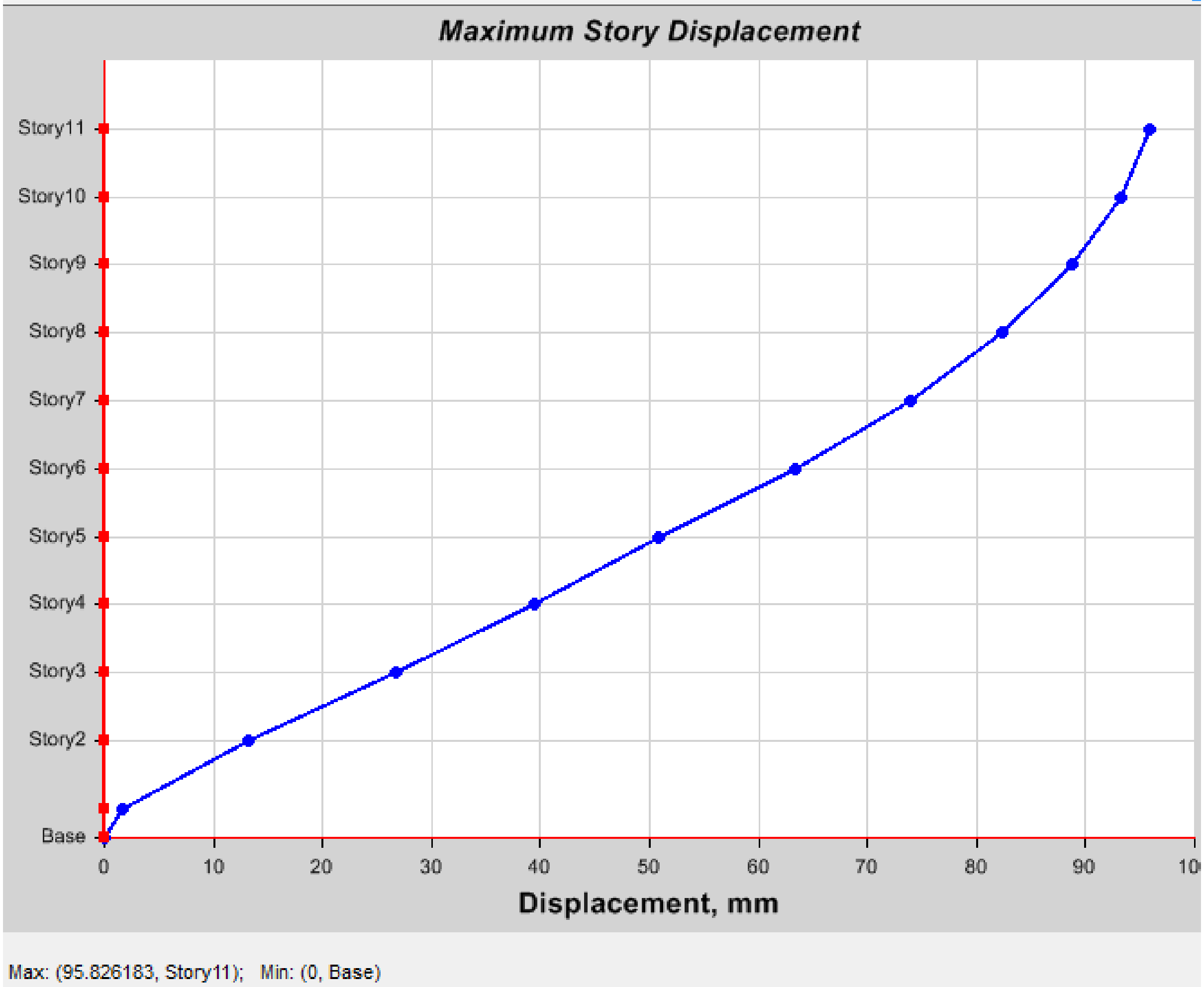


Figure No. 4.1 Max storey displacement for EQx

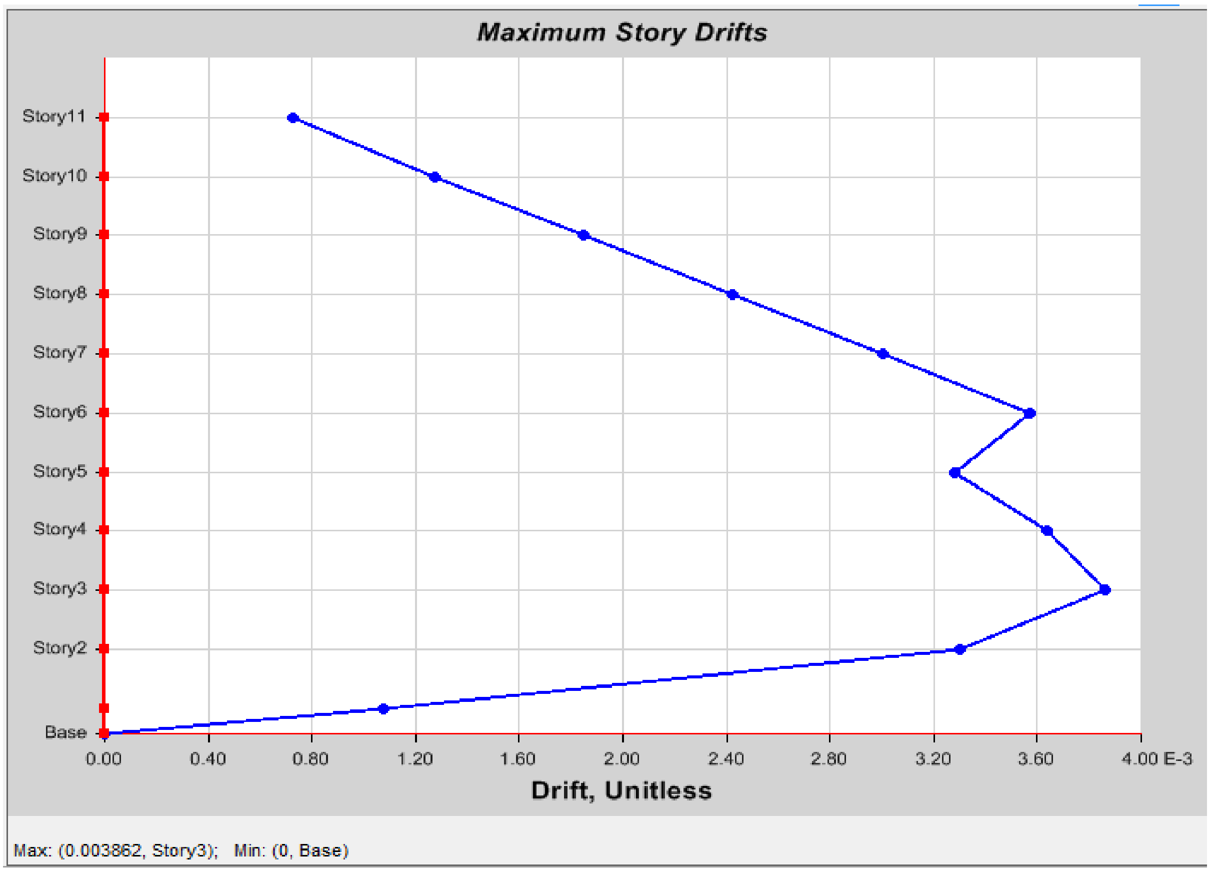


Figure No. 4.2 Max storey drift for EQx

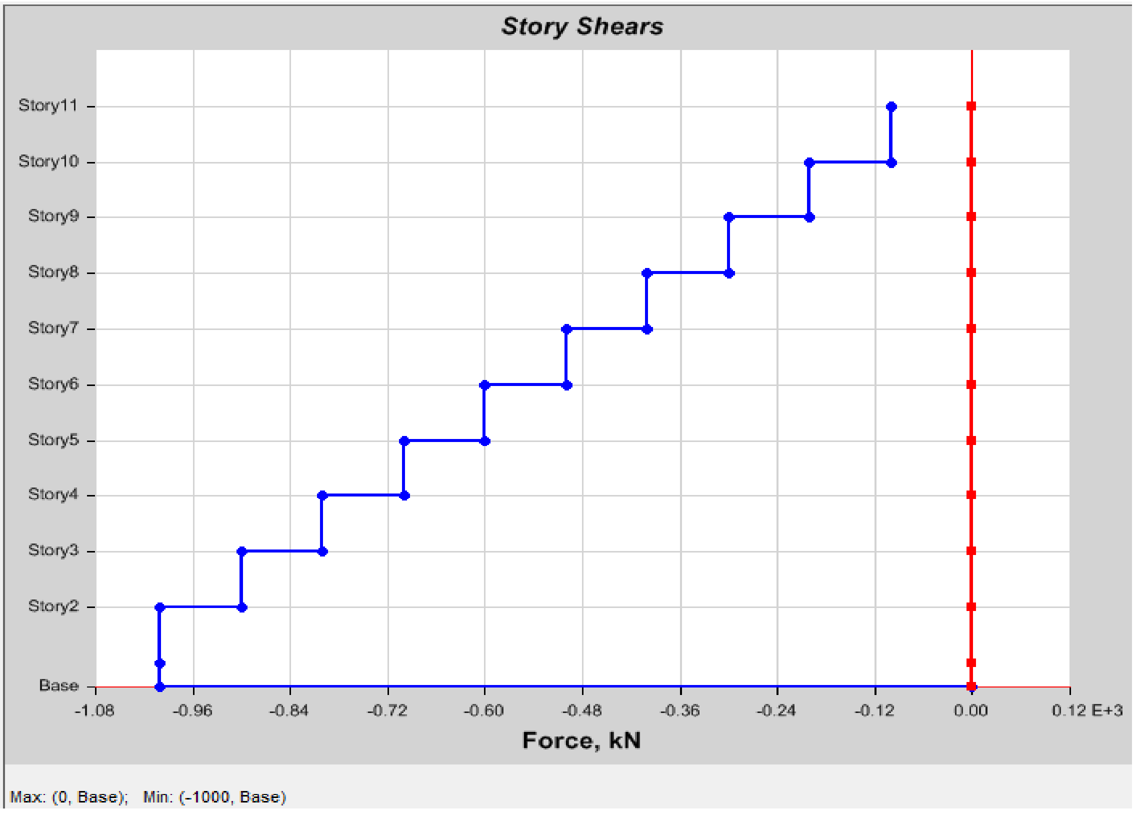


Figure No. 4.3 Storey shear for EQx

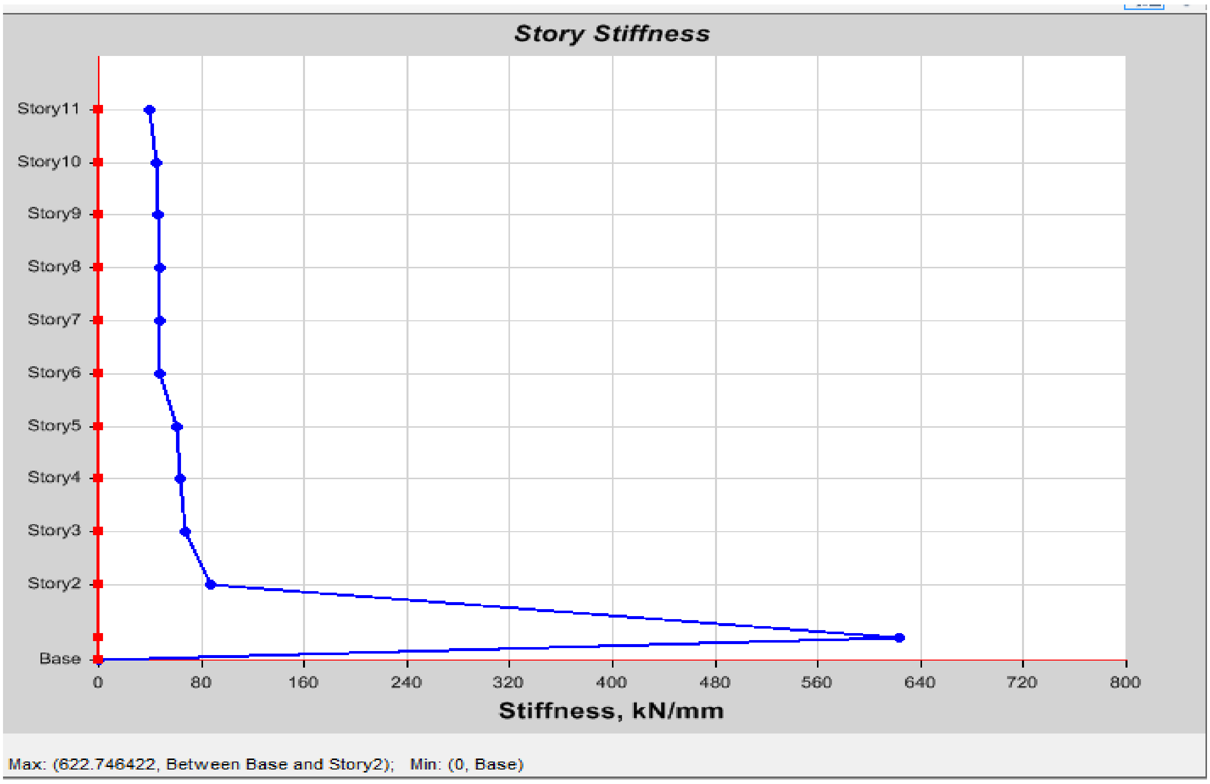


Figure No. 4.4 Storey stiffness for EQx

4.2.2 FRAME 2 M40 grade concrete HYSD 415 STEEL

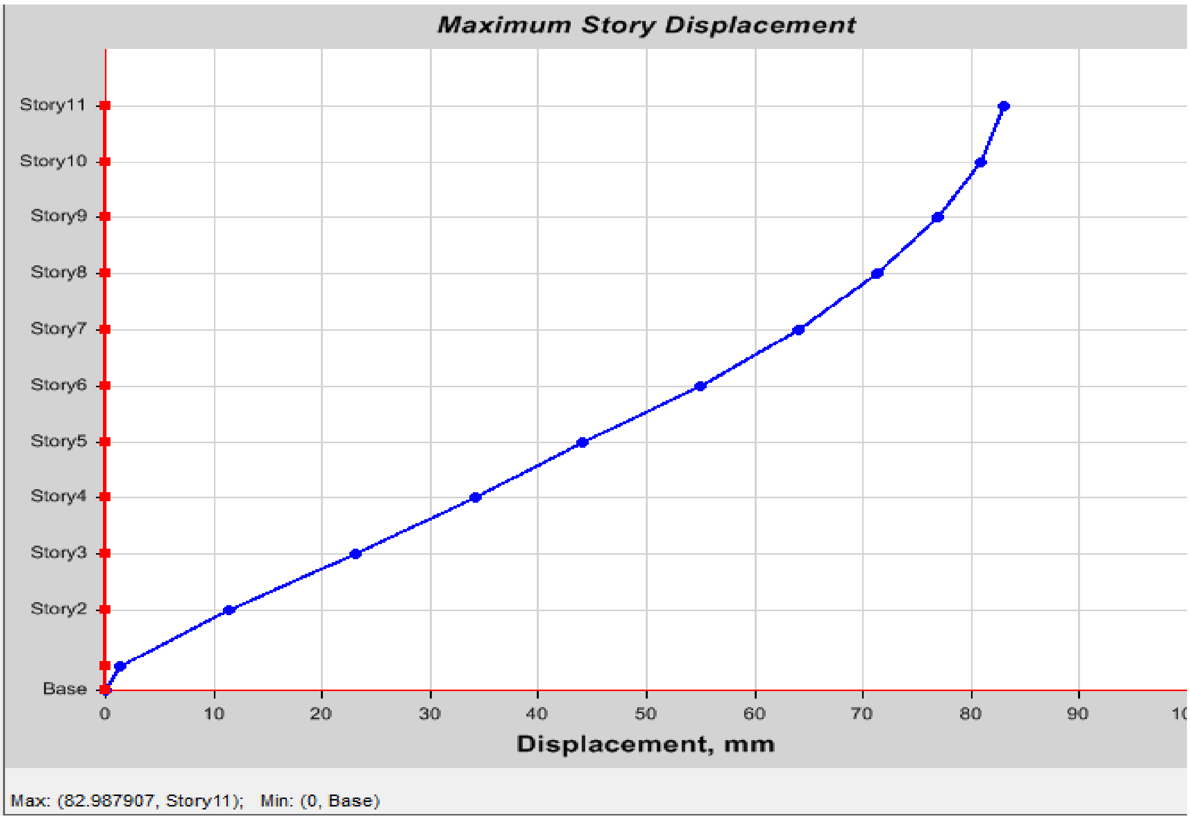


Figure No. 4.5 Max storey displacement for EQx

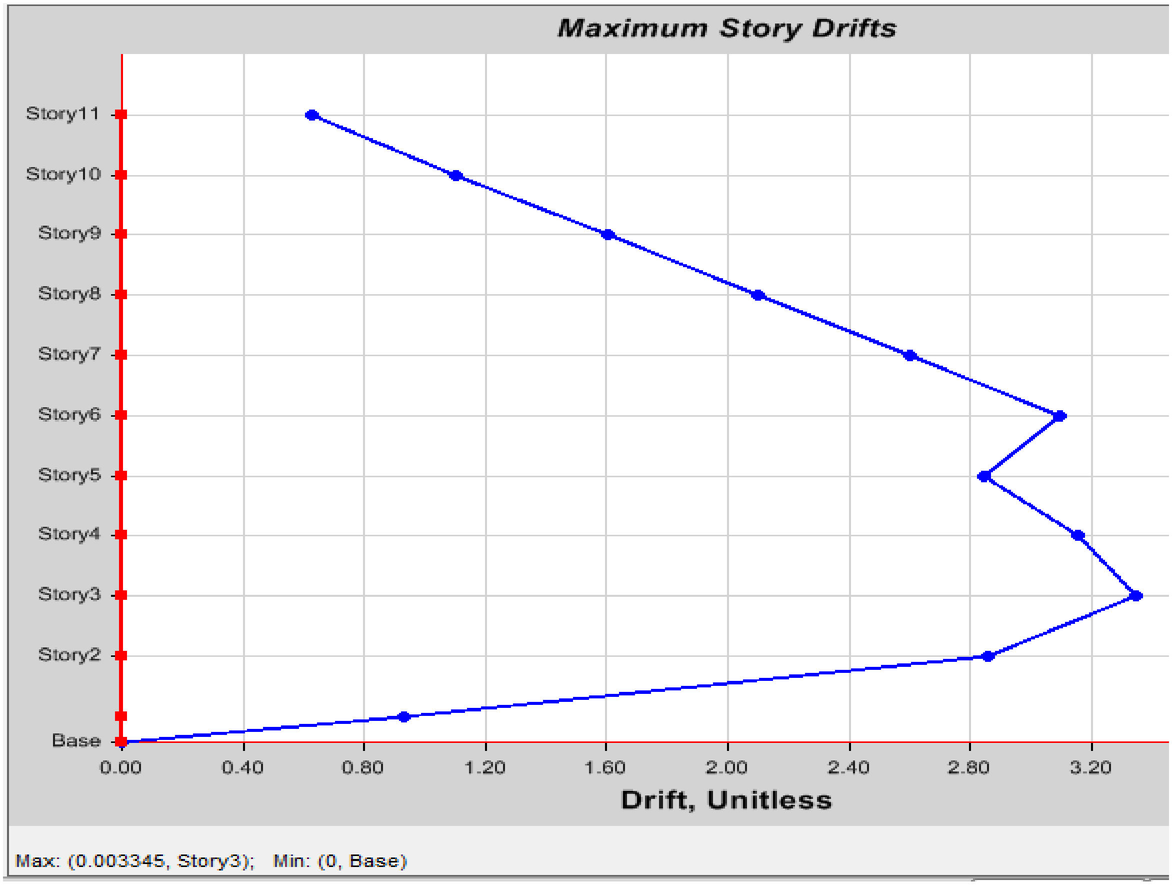


Figure No. 4.6 Max storey drift for EQx

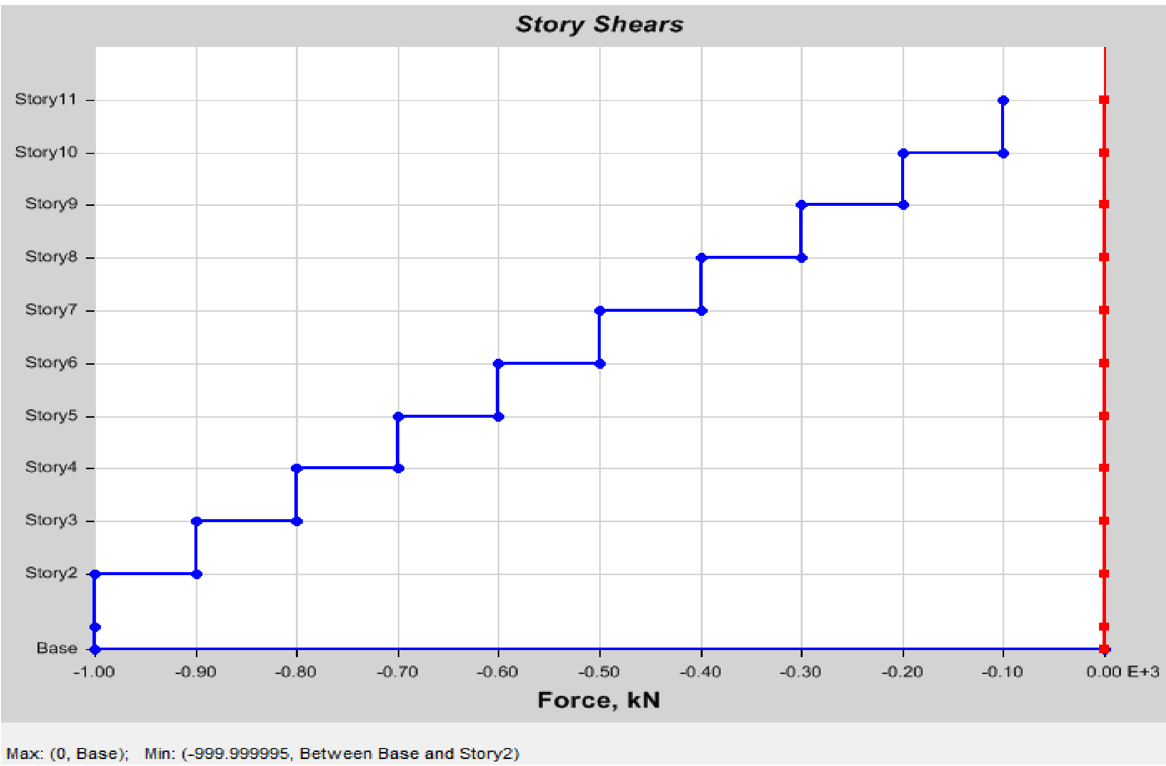


Figure No. 4.7 Storey shears for EQx

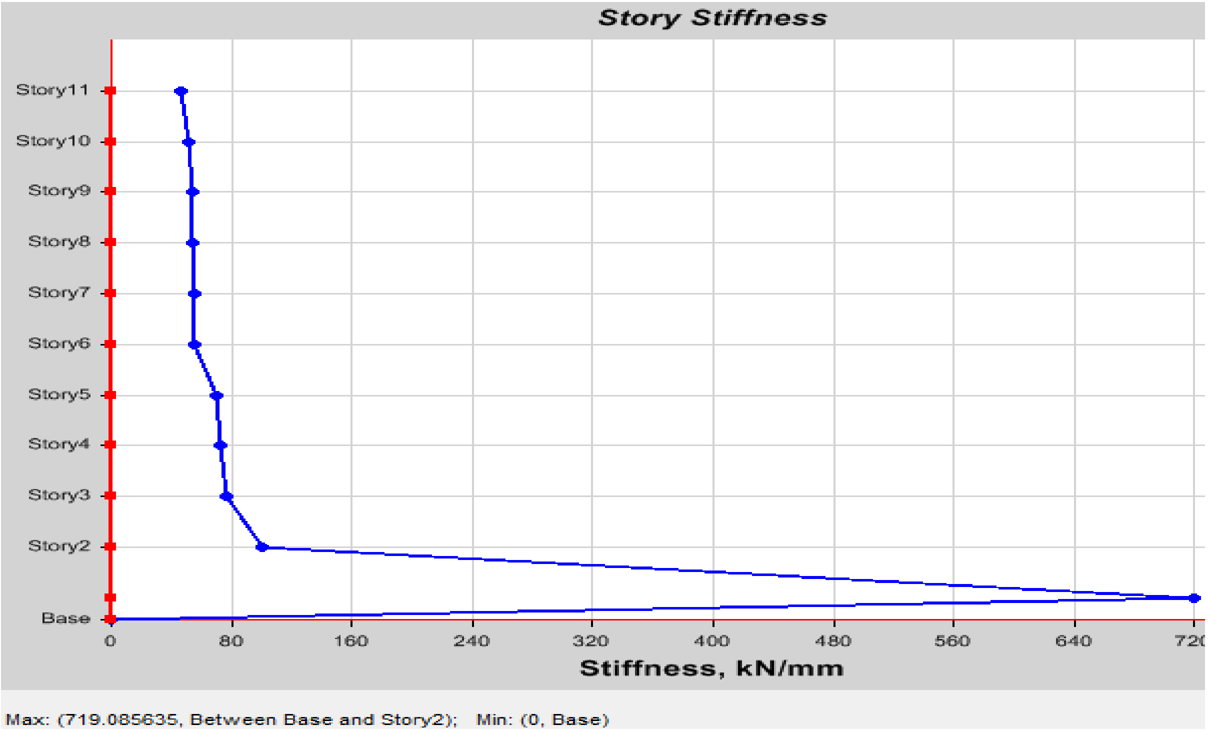


Figure No. 4.8Storey stiffness for EQx

4.2.3 FRAME 3 M30 grade concrete HYSD 500

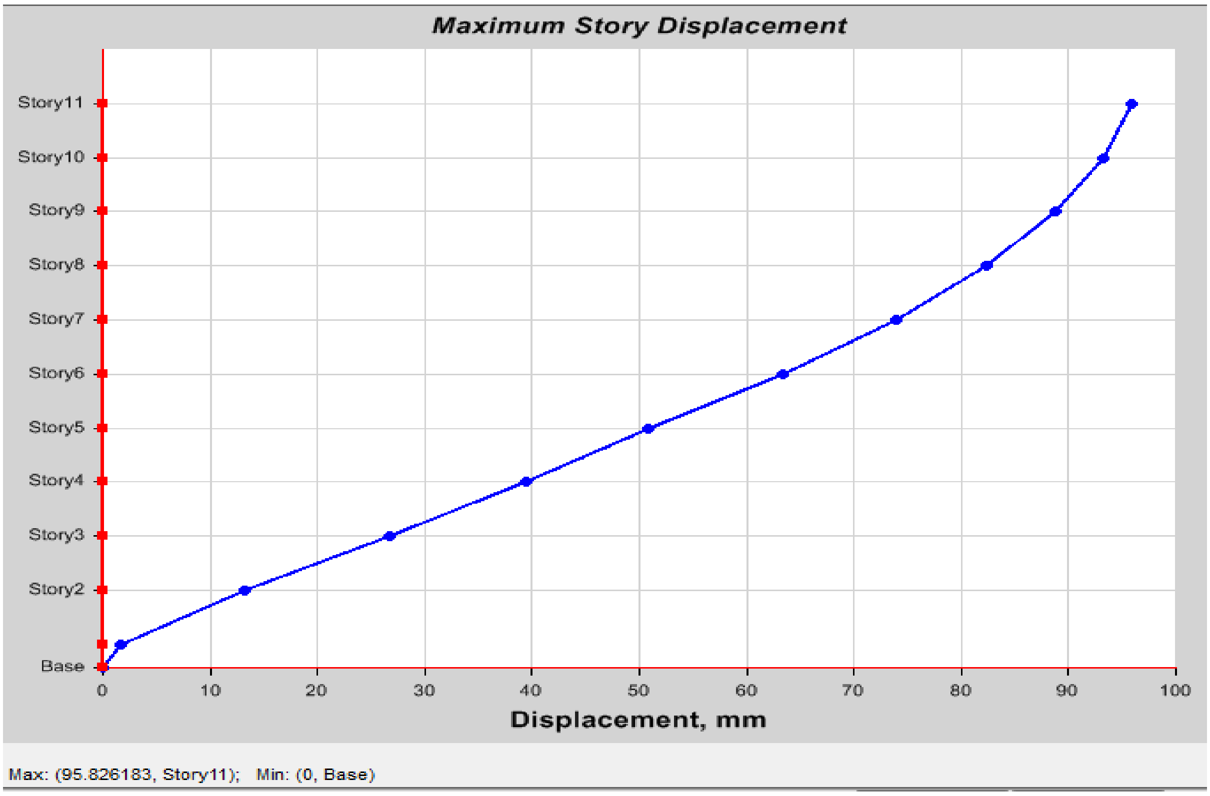


Figure No. 4.9 Max storey displacement for EQx

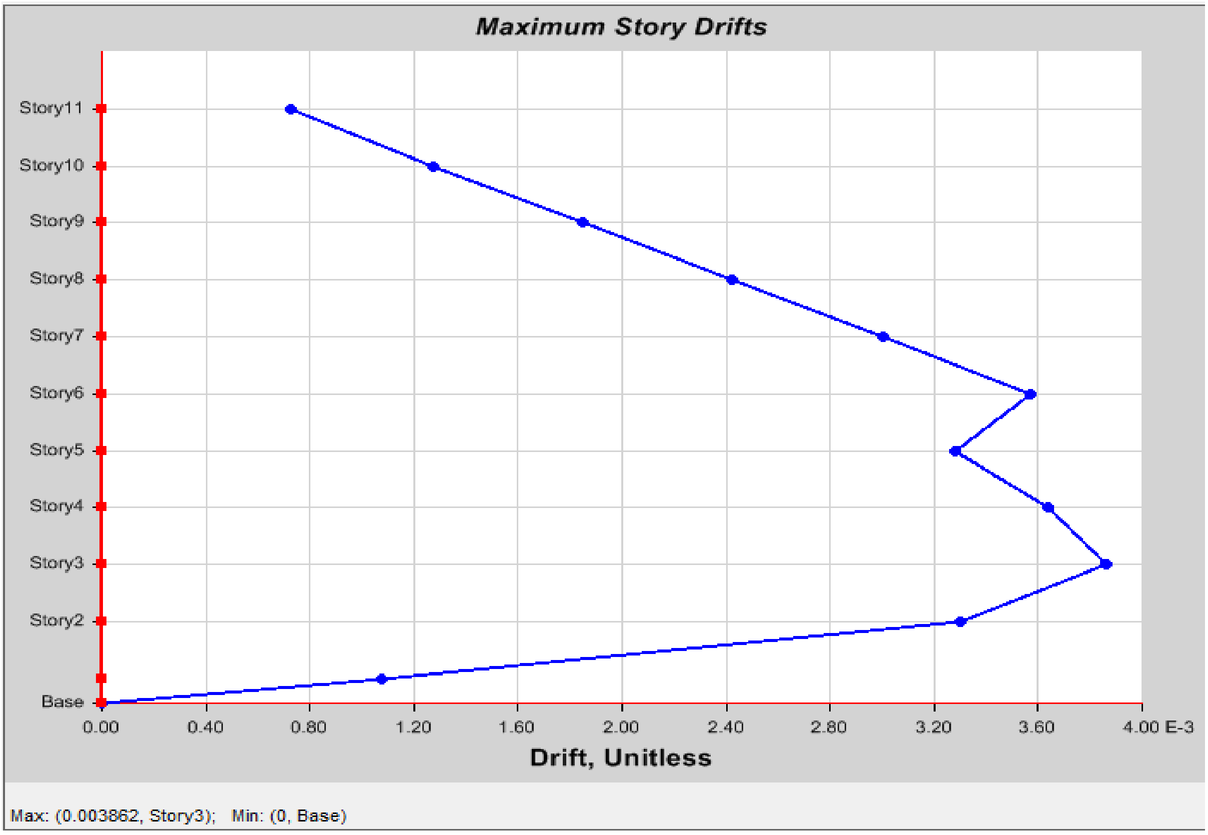


Figure No. 4.10 Maximum storey drift for EQx

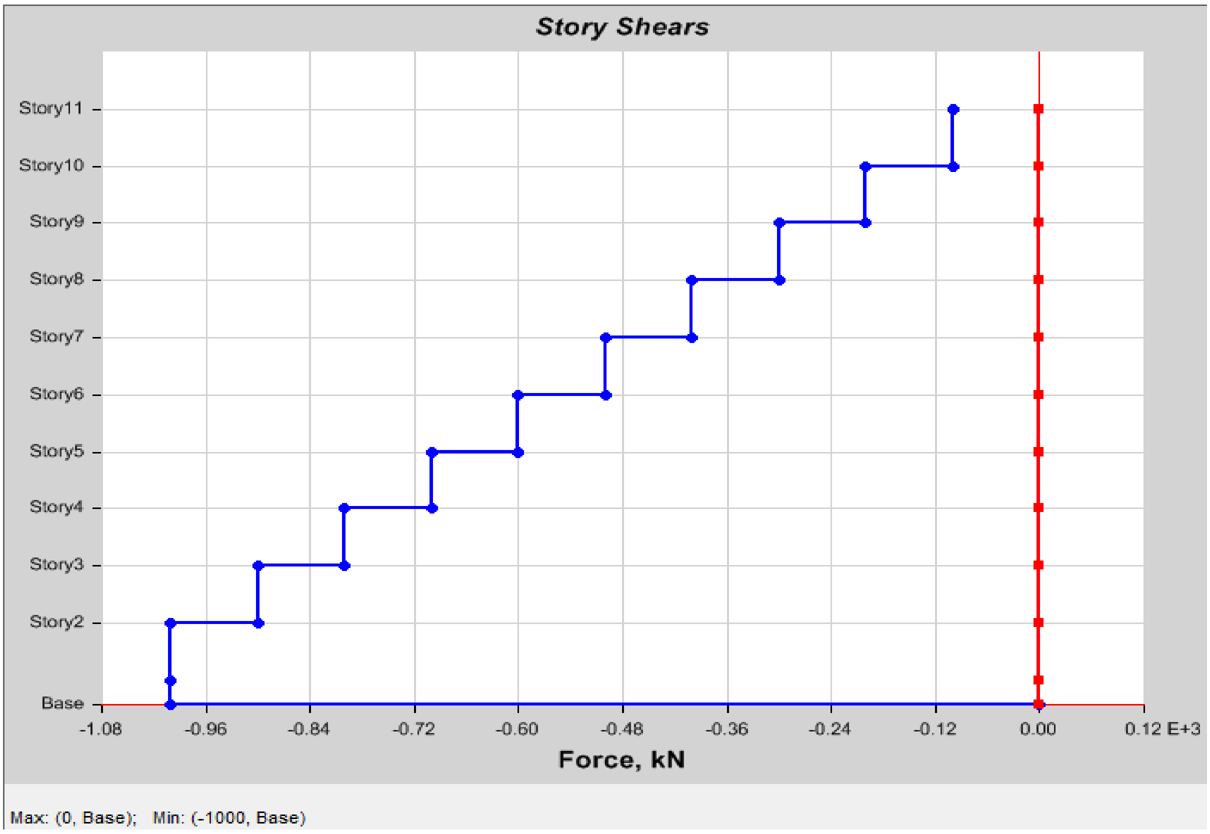


Figure No. 4.11 Storey shear for EQx

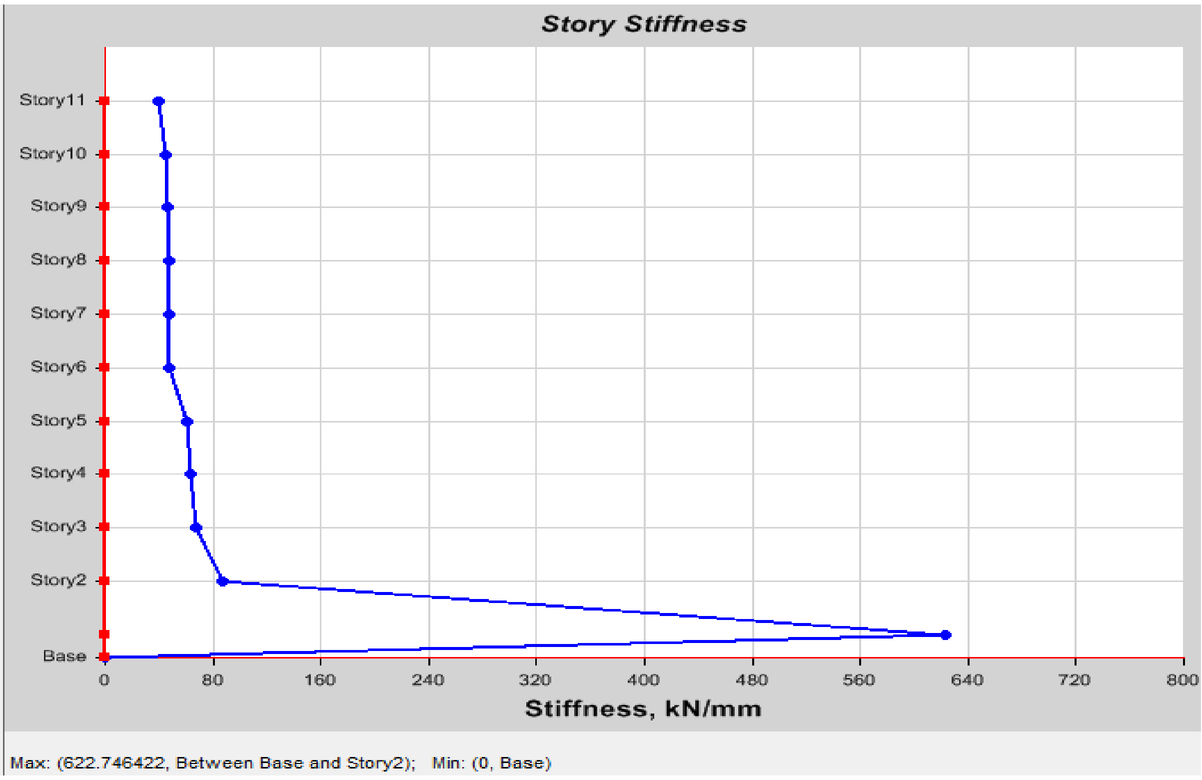


Figure No. 4.12 Storey stiffness for EQx

4.2.4 FRAME 4 M40 grade concrete and Fe500 steel

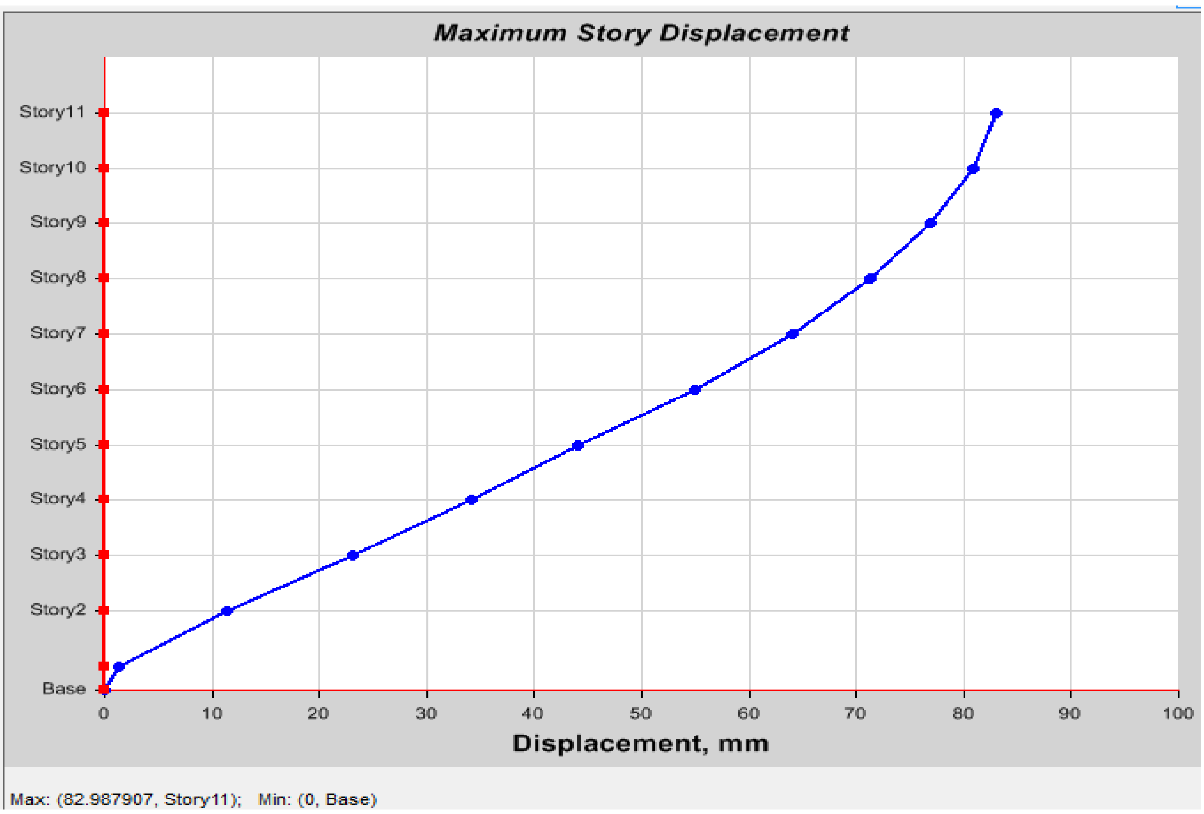


Figure No. 4.13 Max storey displacement

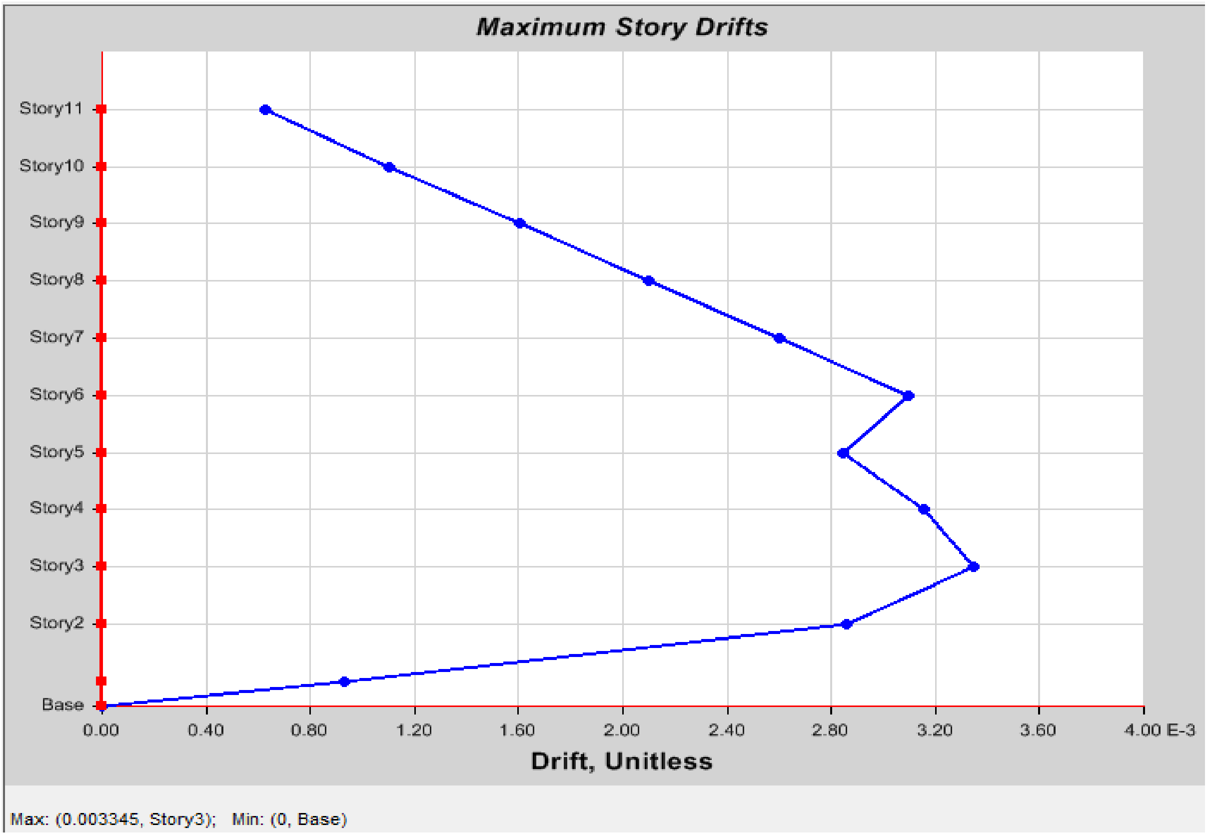


Figure No. 4.14 Max storey drift for EQx

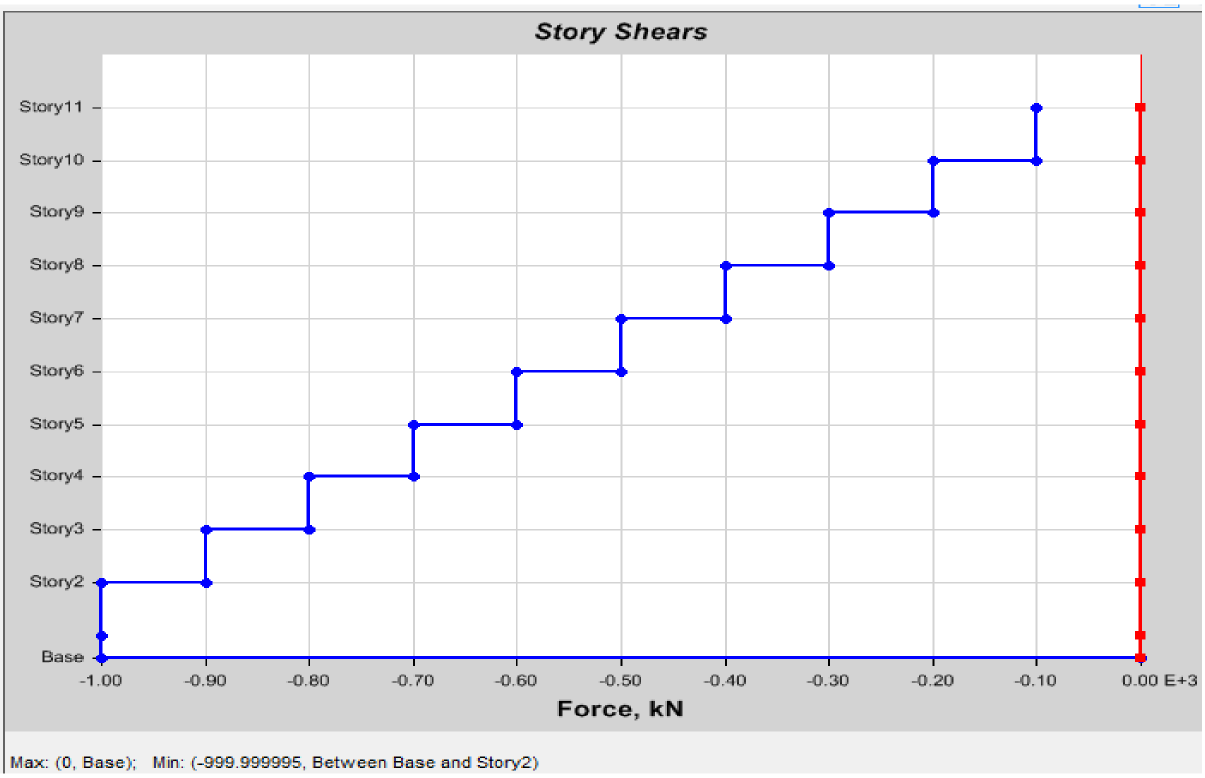


Figure No. 4.15 Storey shear for EQx

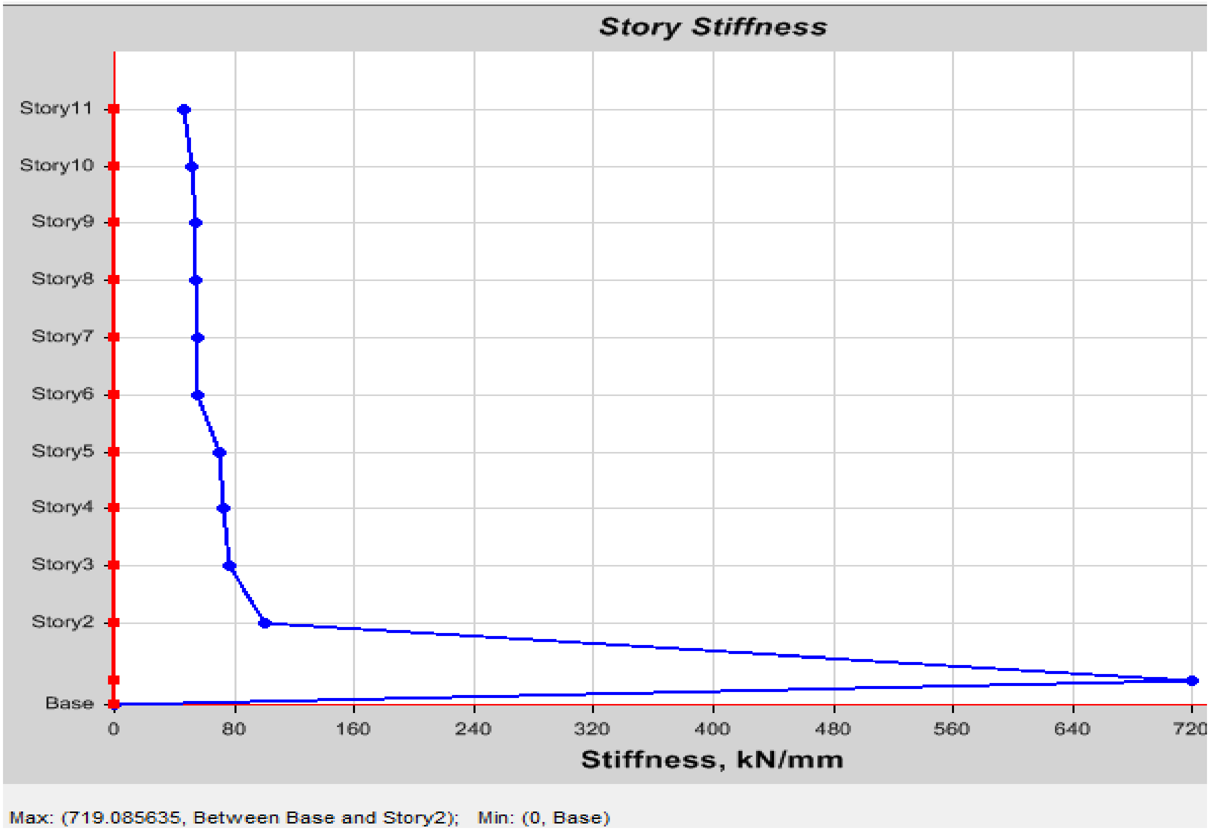


Figure No. 4.16 Storey stiffness for EQx

4.3 MODAL ANALYSIS

4.3.1 INTRODUCTION

Modal analysis, or the mode-superposition method, is a linear dynamic-response procedure which evaluates and superimposes free-vibration mode shapes to characterize displacement patterns. Mode shapes describe the configurations into which a structure will naturally displace. Typically, lateral displacement patterns are of primary concern. Mode shapes of low-order mathematical expression tend to provide the greatest contribution to structural response. As orders increase, mode shapes contribute less, and are predicted less reliably. It is reasonable to truncate analysis when the number of mode shapes is sufficient. Eigenvector analysis determines the undamped free-vibration mode shapes and frequencies of the system. These natural modes provide an excellent insight into the behaviour of the structure.

4.3.2 MODAL PARAMETERS

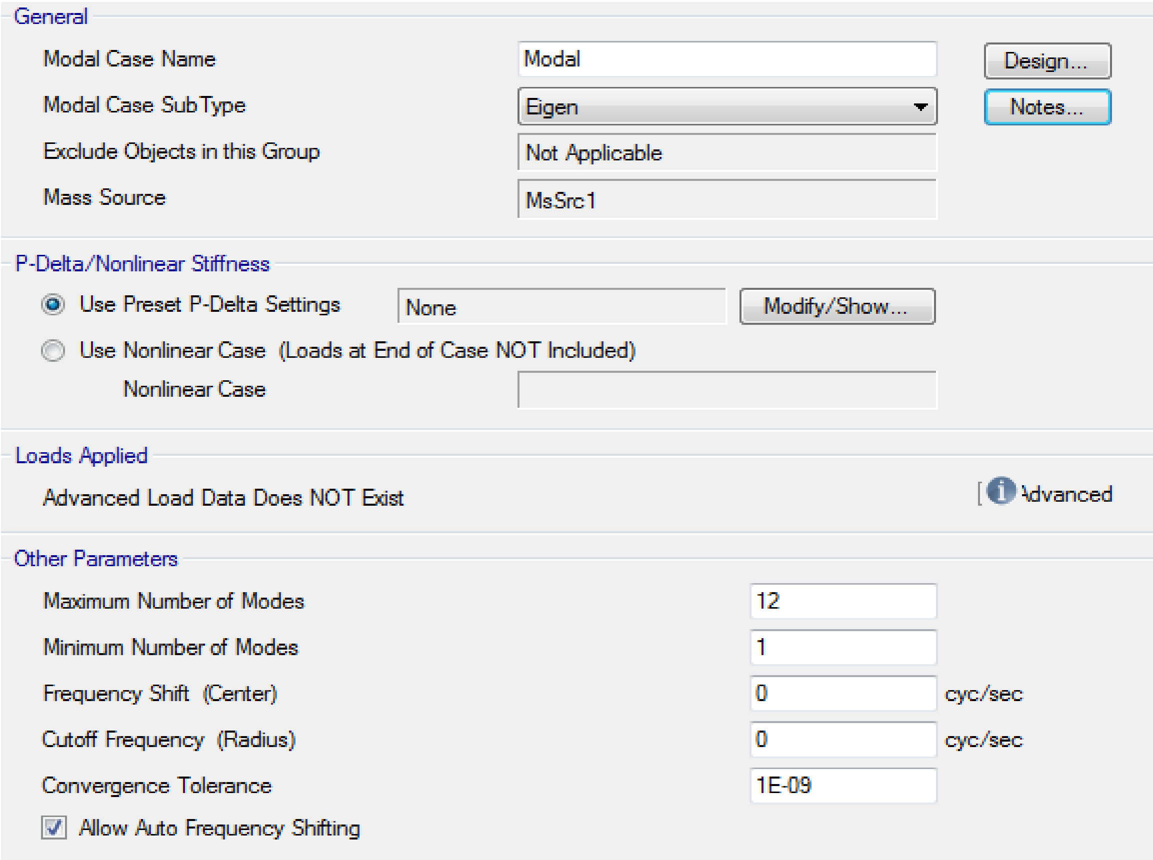


Figure No. 4.17 Modal parameters

4.3.3 MODAL RESULTS

4.3.3.1 FRAME 1

Table 4.1 Modal period and frequencies,F1

Case	Mode	Period sec	Frequency cyc/sec	Circular Frequency rad/sec	Eigenvalue rad^2/sec^2
Modal	1	1.108	0.903	5.6706	32.1555
Modal	2	0.382	2.615	16.4276	269.8652
Modal	3	0.222	4.505	28.3088	801.3908
Modal	4	0.15	6.662	41.8559	1751.9146
Modal	5	0.113	8.846	55.5826	3089.423
Modal	6	0.089	11.22	70.4973	4969.8687
Modal	7	0.072	13.816	86.8104	7536.0375
Modal	8	0.062	16.049	100.8409	10168.892
Modal	9	0.057	17.48	109.8315	12062.9586
Modal	10	0.045	22.187	139.4033	19433.2881
Modal	11	0.027	37.318	234.4737	54977.9387
Modal	12	0.026	38.528	242.0778	58601.6379

Table 4.2 Modal load participating ratio,F1

Case	Item Type	Item	Static %	Dynamic %
Modal	Acceleration	UX	100	94.16
Modal	Acceleration	UY	0	0
Modal	Acceleration	UZ	0	0

Table 4.3 Modal direction factors, F1

Case	Mode	Period sec	UX	UY	UZ	RZ
Modal	1	1.108	1	0	0	0
Modal	2	0.382	1	0	0	0
Modal	3	0.222	1	0	0	0
Modal	4	0.15	1	0	0	0
Modal	5	0.113	1	0	0	0
Modal	6	0.089	1	0	0	0
Modal	7	0.072	1	0	0	0
Modal	8	0.062	1	0	0	0
Modal	9	0.057	1	0	0	0
Modal	10	0.045	1	0	0	0
Modal	11	0.027	1	0	0	0
Modal	12	0.026	1	0	0	0

Table 4.4 Modal participating mass ratios part 1, F1

Case	Mode	Period sec	UX	UY	UZ	Sum UX	Sum UY	Sum UZ
Modal	1	1.108	0.7304	0	0	0.7304	0	0
Modal	2	0.382	0.1141	0	0	0.8445	0	0
Modal	3	0.222	0.0359	0	0	0.8805	0	0
Modal	4	0.15	0.0204	0	0	0.9008	0	0
Modal	5	0.113	0.0145	0	0	0.9153	0	0
Modal	6	0.089	0.008	0	0	0.9233	0	0
Modal	7	0.072	0.0055	0	0	0.9288	0	0
Modal	8	0.062	0.0056	0	0	0.9344	0	0
Modal	9	0.057	0.0026	0	0	0.937	0	0
Modal	10	0.045	0.0046	0	0	0.9416	0	0
Modal	11	0.027	0	0	0	0.9416	0	0
Modal	12	0.026	0	0	0	0.9416	0	0

Table 4.5 Modal participation mass ratios part 2 ,F1

Case	Mode	RX	RY	RZ	Sum RX	Sum RY	Sum RZ
Modal	1	0	0.2754	0	0	0.2754	0
Modal	2	0	0.3824	0	0	0.6578	0
Modal	3	0	0.0426	0	0	0.7004	0

Table 4.6 Modal participation mass ratios contd...,F1

Case	Mode	RX	RY	RZ	Sum RX	Sum RY	Sum RZ
Modal	4	0	0.0625	0	0	0.7628	0
Modal	5	0	0.0287	0	0	0.7915	0
Modal	6	0	0.0219	0	0	0.8134	0
Modal	7	0	0.0109	0	0	0.8243	0
Modal	8	0	0.0132	0	0	0.8375	0
Modal	9	0	0.0056	0	0	0.8431	0
Modal	10	0	0.0114	0	0	0.8544	0
Modal	11	0	0	0	0	0.8544	0
Modal	12	0	0	0	0	0.8544	0

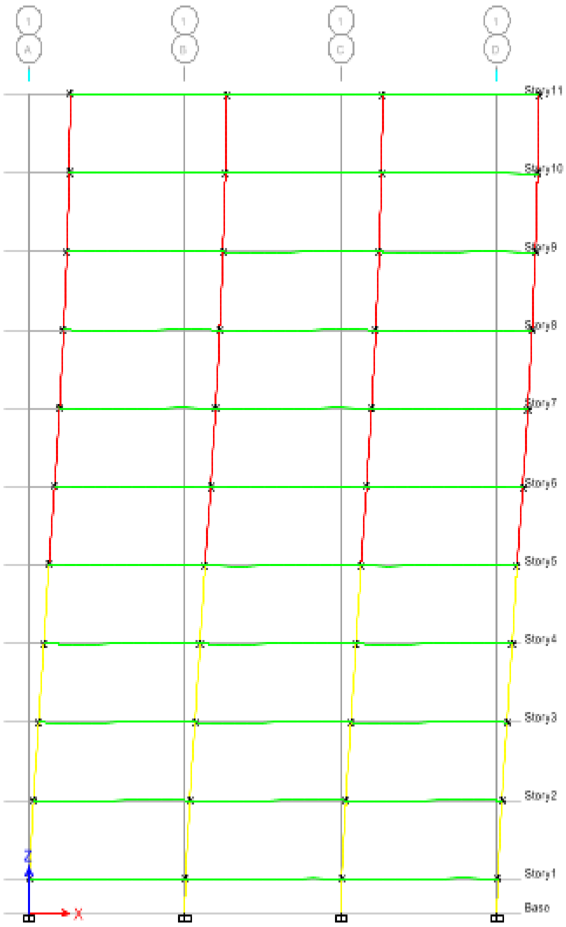


Figure No. 4.18 Mode shape 1

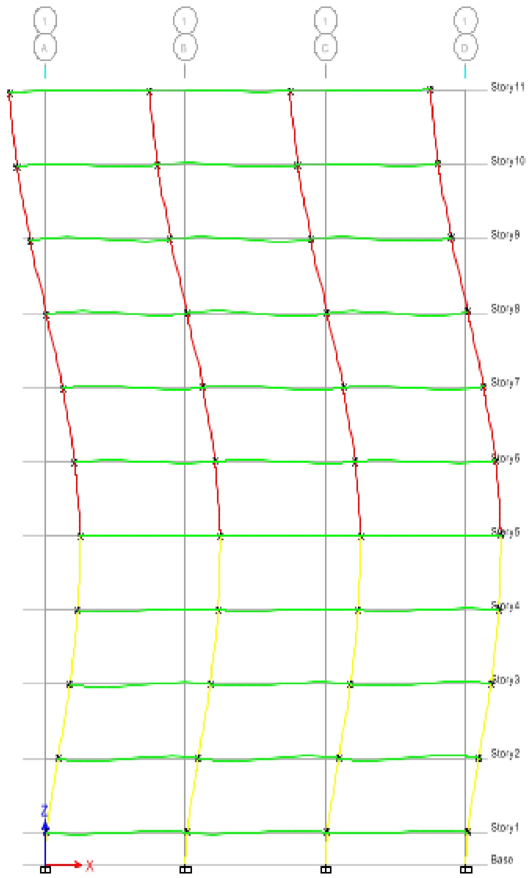


Figure No. 4.19 Mode shape 2

4.3.3.2 FRAME 2

Table 4.7 Modal period and frequencies, F2

Case	Mode	Period sec	Frequency cyc/sec	Circular Frequency rad/sec	Eigenvalue rad²/sec²
Modal	1	1.031	0.97	6.0934	37.13
Modal	2	0.356	2.809	17.6526	311.6134
Modal	3	0.207	4.841	30.4198	925.3664
Modal	4	0.14	7.158	44.9771	2022.9368
Modal	5	0.105	9.506	59.7274	3567.3585
Modal	6	0.083	12.057	75.7543	5738.7101
Modal	7	0.067	14.847	93.2838	8701.8666
Modal	8	0.058	17.246	108.3606	11742.0252
Modal	9	0.053	18.784	118.0216	13929.1049

Table 4.8 Modal period and frequencies , F2

Case	Mode	Period sec	Frequency cyc/sec	Circular Frequency rad/sec	Eigenvalue rad²/sec²
Modal	10	0.042	23.841	149.7986	22439.6285
Modal	11	0.025	40.1	251.9584	63483.0559
Modal	12	0.024	41.401	260.1295	67667.3434

Table 4.9 Modal load participating ratio , F2

Case	Item Type	Item	Static %	Dynamic %
Modal	Acceleration	UX	100	94.16
Modal	Acceleration	UY	0	0
Modal	Acceleration	UZ	0	0

Table 4.10 Modal participating mass ratios part 1 , F2

Case	Mode	Period sec	UX	UY	UZ	Sum UX	Sum UY	Sum UZ
Modal	1	1.031	0.7304	0	0	0.7304	0	0
Modal	2	0.356	0.1141	0	0	0.8445	0	0
Modal	3	0.207	0.0359	0	0	0.8805	0	0
Modal	4	0.14	0.0204	0	0	0.9008	0	0
Modal	5	0.105	0.0145	0	0	0.9153	0	0
Modal	6	0.083	0.008	0	0	0.9233	0	0
Modal	7	0.067	0.0055	0	0	0.9288	0	0
Modal	8	0.058	0.0056	0	0	0.9344	0	0
Modal	9	0.053	0.0026	0	0	0.937	0	0
Modal	10	0.042	0.0046	0	0	0.9416	0	0
Modal	11	0.025	0	0	0	0.9416	0	0
Modal	12	0.024	0	0	0	0.9416	0	0

Table 4.11 Modal participating mass ratios part 2,F2

Case	Mode	RX	RY	RZ	Sum RX	Sum RY	Sum RZ
Modal	1	0	0.2754	0	0	0.2754	0
Modal	2	0	0.3824	0	0	0.6578	0
Modal	3	0	0.0426	0	0	0.7004	0
Modal	4	0	0.0625	0	0	0.7628	0
Modal	5	0	0.0287	0	0	0.7915	0
Modal	6	0	0.0219	0	0	0.8134	0
Modal	7	0	0.0109	0	0	0.8243	0
Modal	8	0	0.0132	0	0	0.8375	0
Modal	9	0	0.0056	0	0	0.8431	0
Modal	10	0	0.0114	0	0	0.8544	0
Modal	11	0	0	0	0	0.8544	0
Modal	12	0	0	0	0	0.8544	0

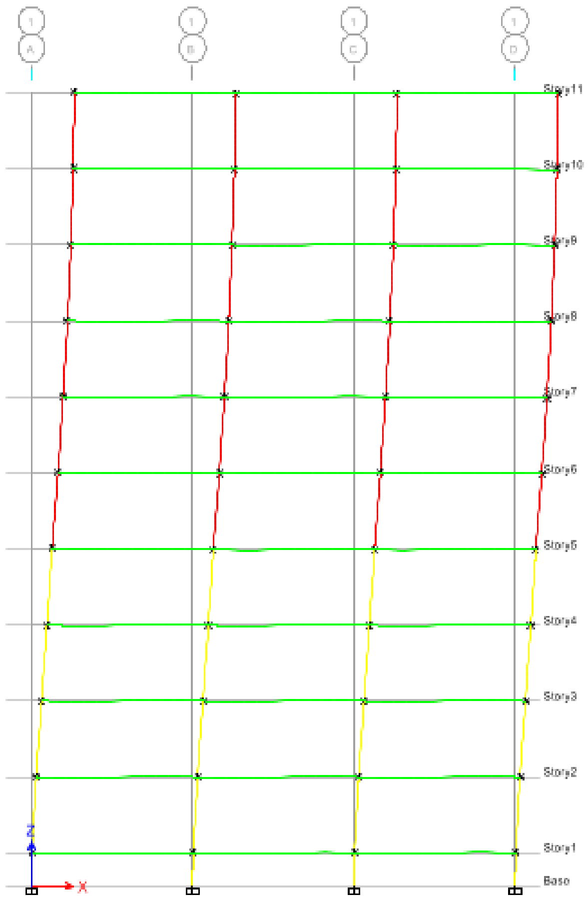


Figure No. 4.20 Mode shape 1

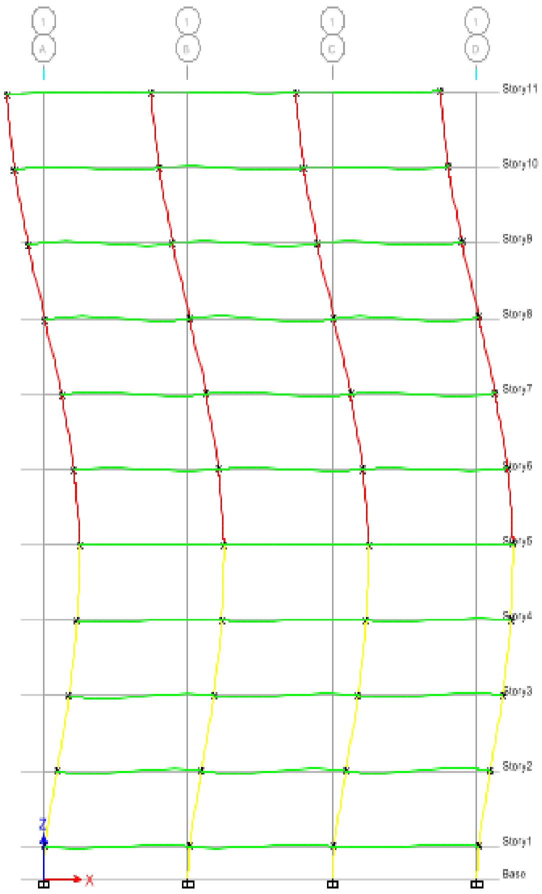


Figure No. 4.21 Mode shape 2

4.3.3.3 Frame 3

Table 4.12 Modal period and frequencies, F3

Case	Mode	Period sec	Frequency cyc/sec	Circular Frequency rad/sec	Eigenvalue rad²/sec²
Modal	1	1.108	0.903	5.6706	32.1555
Modal	2	0.382	2.615	16.4276	269.8652
Modal	3	0.222	4.505	28.3088	801.3908
Modal	4	0.15	6.662	41.8559	1751.9146
Modal	5	0.113	8.846	55.5826	3089.423
Modal	6	0.089	11.22	70.4973	4969.8687
Modal	7	0.072	13.816	86.8104	7536.0375
Modal	8	0.062	16.049	100.8409	10168.892
Modal	9	0.057	17.48	109.8315	12062.9586
Modal	10	0.045	22.187	139.4033	19433.2881
Modal	11	0.027	37.318	234.4737	54977.9387
Modal	12	0.026	38.528	242.0778	58601.6379

Table 4.13 Modal load participating ratio, F3

Case	Item Type	Item	Static %	Dynamic %
Modal	Acceleration	UX	100	94.16
Modal	Acceleration	UY	0	0
Modal	Acceleration	UZ	0	0

Table 4.14 Modal participating mass ratios , F3

Case	Mode	Period sec	UX	UY	UZ	Sum UX	Sum UY	Sum UZ
Modal	1	1.108	0.7304	0	0	0.7304	0	0
Modal	2	0.382	0.1141	0	0	0.8445	0	0
Modal	3	0.222	0.0359	0	0	0.8805	0	0
Modal	4	0.15	0.0204	0	0	0.9008	0	0
Modal	5	0.113	0.0145	0	0	0.9153	0	0
Modal	6	0.089	0.008	0	0	0.9233	0	0
Modal	7	0.072	0.0055	0	0	0.9288	0	0
Modal	8	0.062	0.0056	0	0	0.9344	0	0
Modal	9	0.057	0.0026	0	0	0.937	0	0
Modal	10	0.045	0.0046	0	0	0.9416	0	0
Modal	11	0.027	0	0	0	0.9416	0	0
Modal	12	0.026	0	0	0	0.9416	0	0

Table 4.15 Modal participating mass ratios part 2, F3

Case	Mode	RX	RY	RZ	Sum RX	Sum RY	Sum RZ
Modal	1	0	0.2754	0	0	0.2754	0
Modal	2	0	0.3824	0	0	0.6578	0
Modal	3	0	0.0426	0	0	0.7004	0

Case	Mode	RX	RY	RZ	Sum RX	Sum RY	Sum RZ
Modal	4	0	0.0625	0	0	0.7628	0
Modal	5	0	0.0287	0	0	0.7915	0
Modal	6	0	0.0219	0	0	0.8134	0
Modal	7	0	0.0109	0	0	0.8243	0
Modal	8	0	0.0132	0	0	0.8375	0
Modal	9	0	0.0056	0	0	0.8431	0
Modal	10	0	0.0114	0	0	0.8544	0
Modal	11	0	0	0	0	0.8544	0
Modal	12	0	0	0	0	0.8544	0

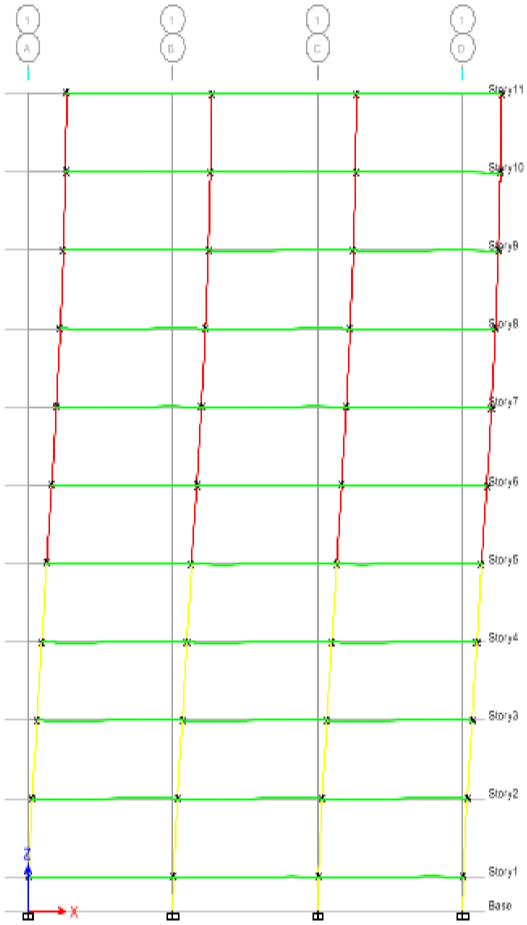


Figure No. 4.22 Mode shape 1

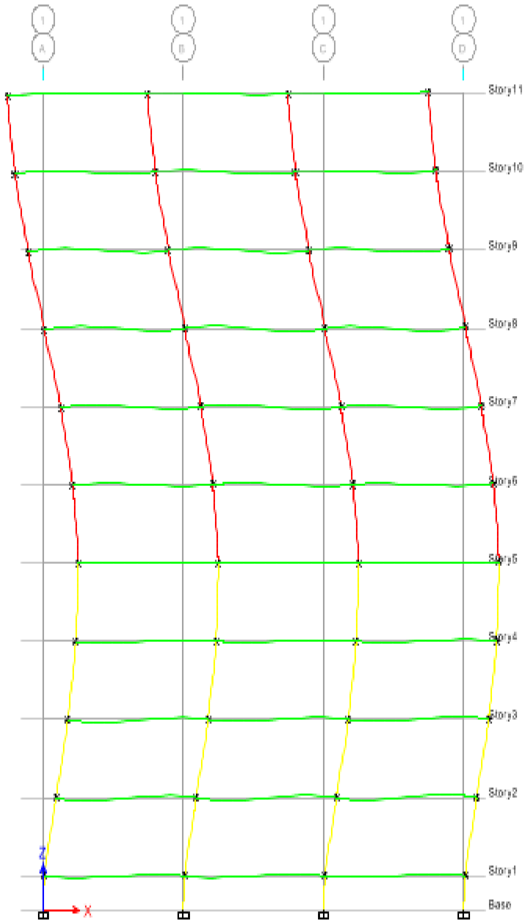


Figure No. 4.23 Mode shape 2

4.3.3.4 Frame 4

Table 4.16 Modal period and frequencies, F4

Case	Mode	Period sec	Frequency cyc/sec	Circular Frequency rad/sec	Eigenvalue rad²/sec²
Modal	1	0.971	1.03	6.4689	41.847
Modal	2	0.335	2.986	18.7636	352.0745
Modal	3	0.194	5.145	32.3269	1045.0258
Modal	4	0.131	7.606	47.791	2283.9774
Modal	5	0.099	10.104	63.4826	4030.0358
Modal	6	0.078	12.811	80.4954	6479.5095
Modal	7	0.063	15.776	99.1247	9825.7087
Modal	8	0.055	18.333	115.1922	13269.2386
Modal	9	0.05	19.956	125.3895	15722.5306
Modal	10	0.039	25.391	159.5384	25452.5053
Modal	11	0.023	42.823	269.066	72396.5301
Modal	12	0.023	44.082	276.9732	76714.1633

Table 4.17 Modal load participating ratio , F4

Case	Item Type	Item	Static %	Dynamic %
Modal	Acceleration	UX	100	99.97
Modal	Acceleration	UY	0	0
Modal	Acceleration	UZ	0	0

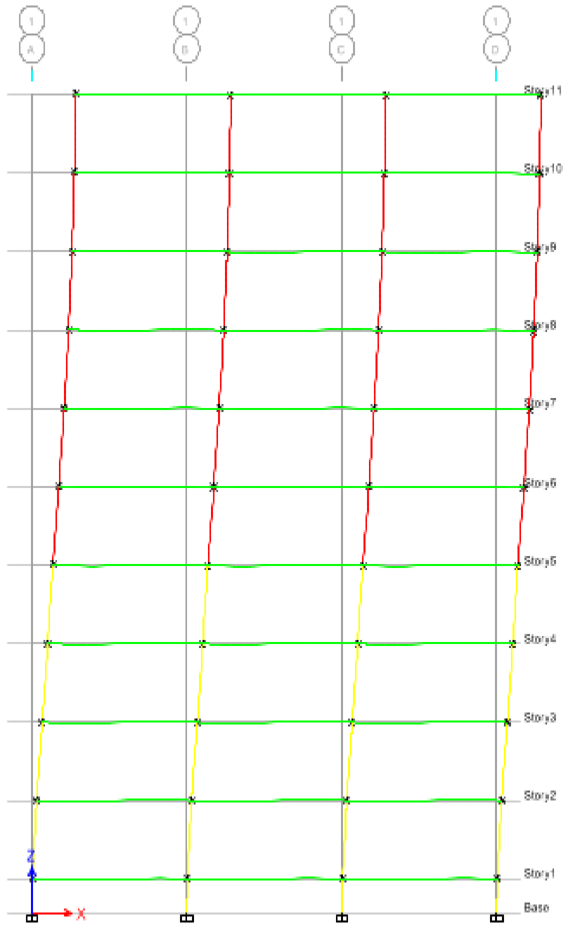
Table 4.18 Modal participating mass ratios part1, F4

Case	Mode	Period sec	UX	UY	UZ	Sum UX	Sum UY	Sum UZ
Modal	1	0.971	0.7309	0	0	0.7309	0	0
Modal	2	0.335	0.1136	0	0	0.8445	0	0
Modal	3	0.194	0.0359	0	0	0.8804	0	0
Modal	4	0.131	0.0203	0	0	0.9007	0	0
Modal	5	0.099	0.0145	0	0	0.9152	0	0
Modal	6	0.078	0.008	0	0	0.9232	0	0
Modal	7	0.063	0.0054	0	0	0.9287	0	0
Modal	8	0.055	0.0056	0	0	0.9343	0	0
Modal	9	0.05	0.0027	0	0	0.937	0	0
Modal	10	0.039	0.0046	0	0	0.9416	0	0
Modal	11	0.023	0	0	0	0.9416	0	0
Modal	12	0.023	0.0581	0	0	0.9997	0	0

Table 4.19 Modal participating mass ratios part2, F4

Case	Mode	RX	RY	RZ	Sum RX	Sum RY	Sum RZ
Modal	1	0	0.2743	0	0	0.2743	0
Modal	2	0	0.3821	0	0	0.6565	0
Modal	3	0	0.0428	0	0	0.6993	0

Case	Mode	RX	RY	RZ	Sum RX	Sum RY	Sum RZ
Modal	4	0	0.0624	0	0	0.7617	0
Modal	5	0	0.0288	0	0	0.7905	0
Modal	6	0	0.022	0	0	0.8125	0
Modal	7	0	0.0109	0	0	0.8234	0
Modal	8	0	0.0133	0	0	0.8367	0
Modal	9	0	0.0058	0	0	0.8425	0
Modal	10	0	0.0115	0	0	0.8539	0
Modal	11	0	0	0	0	0.8539	0
Modal	12	0	0.1452	0	0	0.9991	0



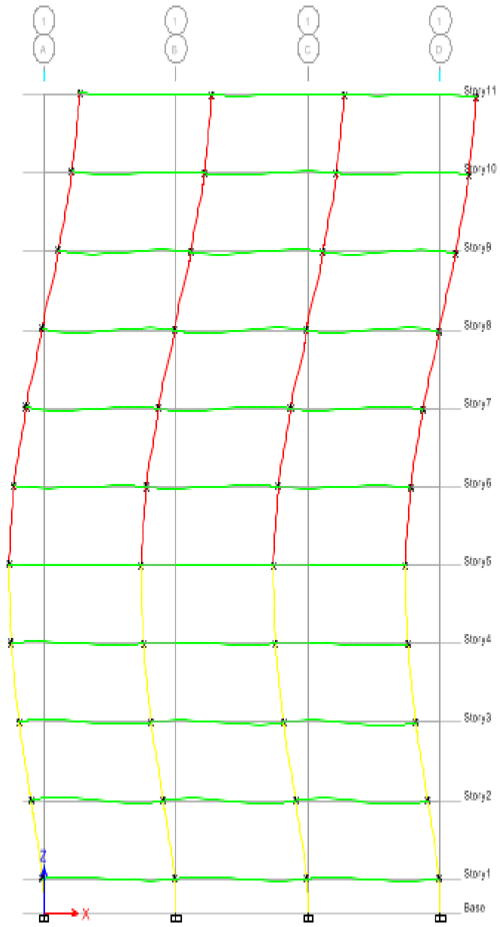


Figure No. 4.24 Mode shape 1

Figure No. 4.25 Mode shape 2

4.4 Reinforcements

4.4.1 FRAME1

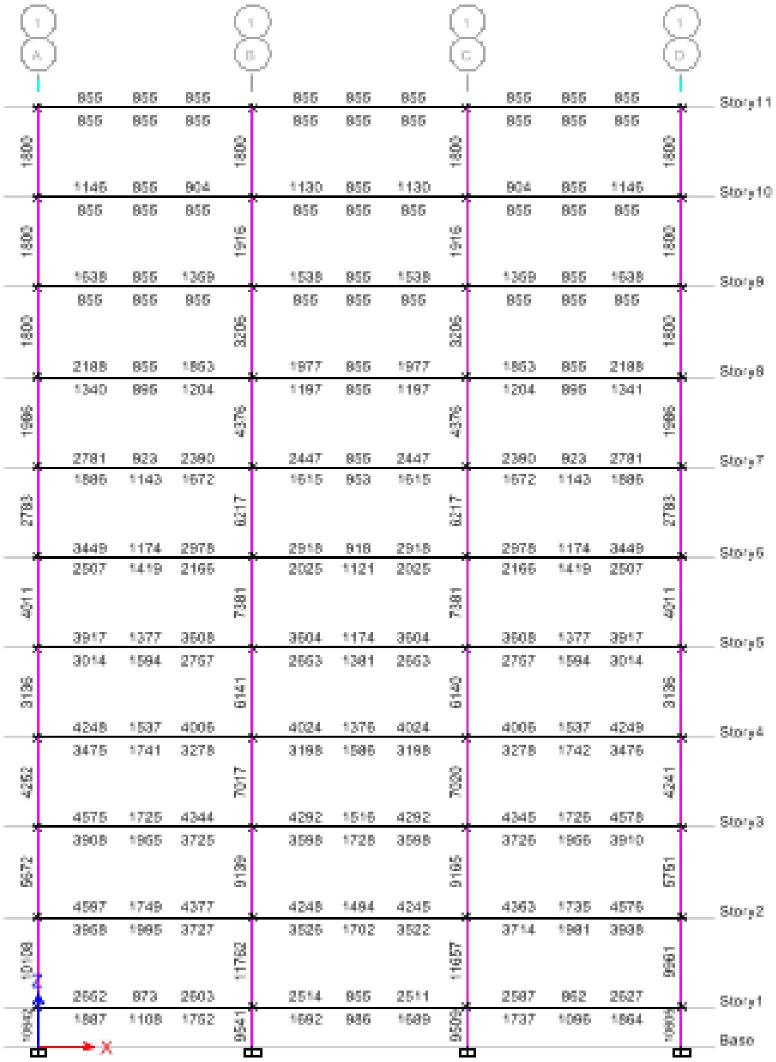


Figure No. 4.26 Longitudinal reinforcement

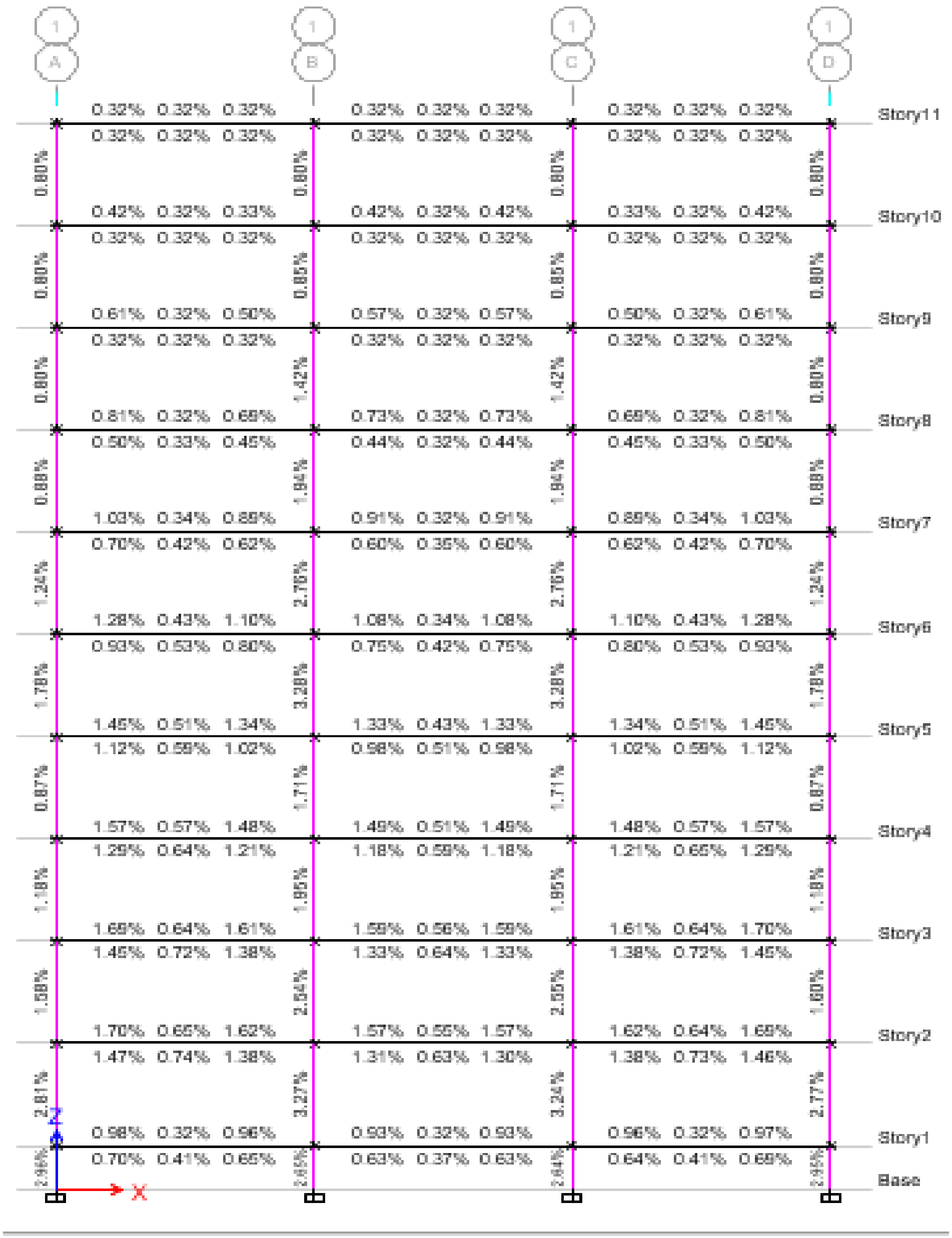


Figure 4.27 Rebar percentage

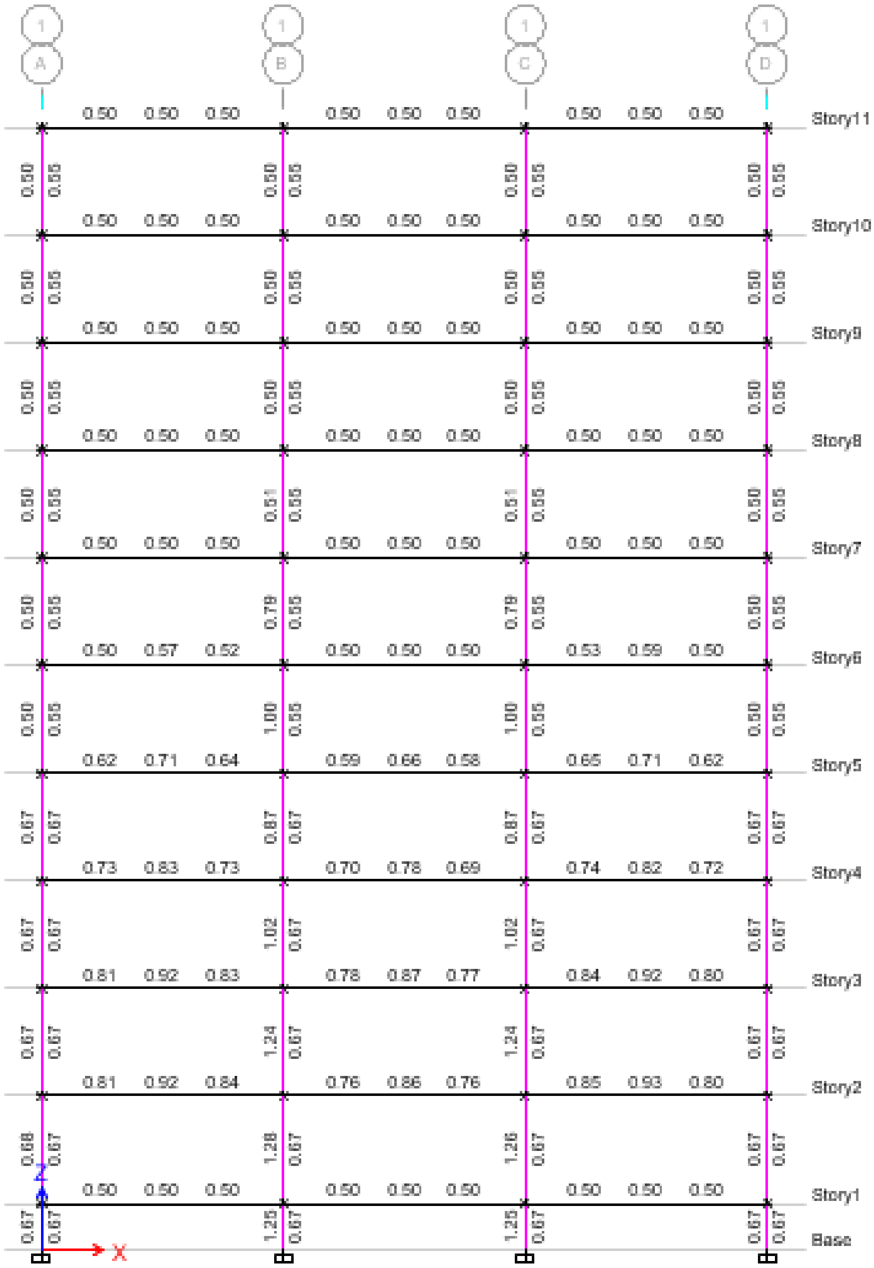


Figure No. 4.28Shear reinforcing details

FRAME 2

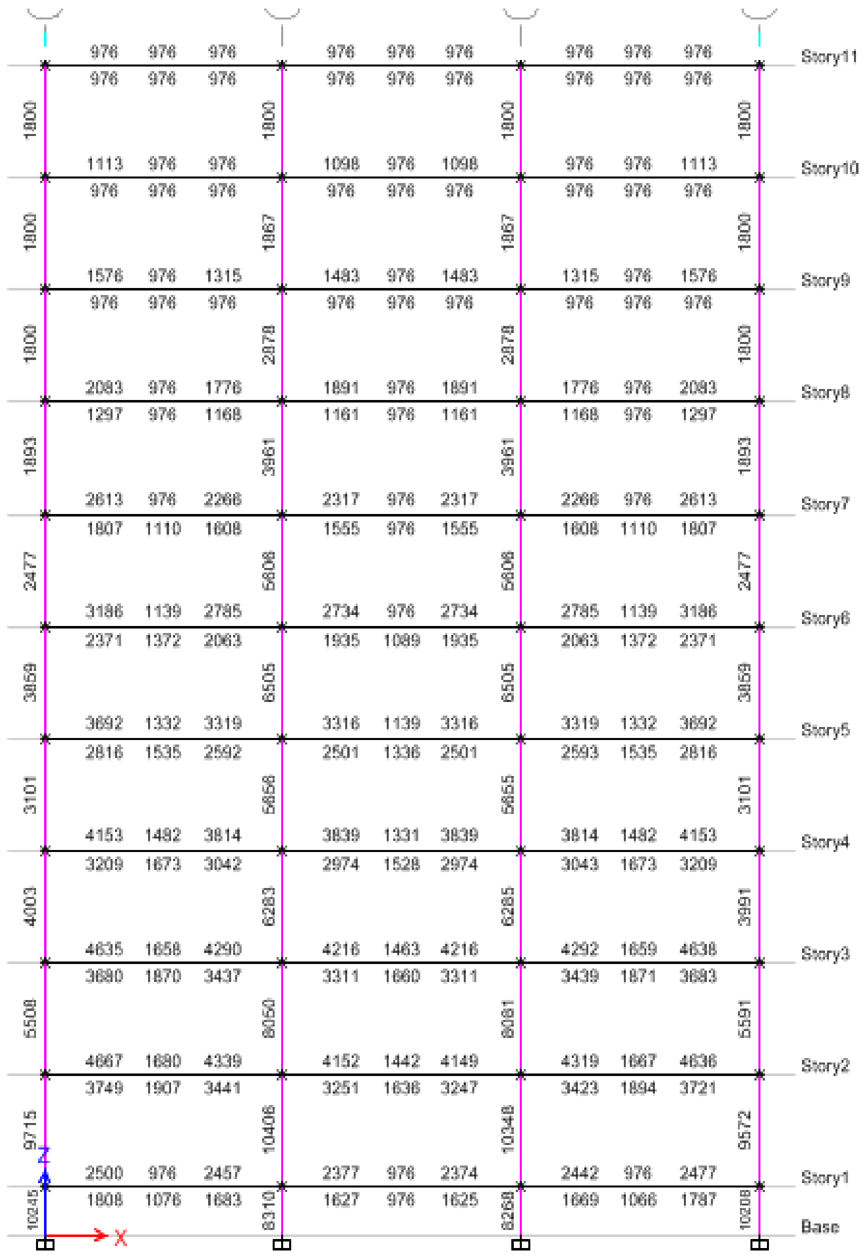


Figure No. 4.29 Longitudinal reinforcement

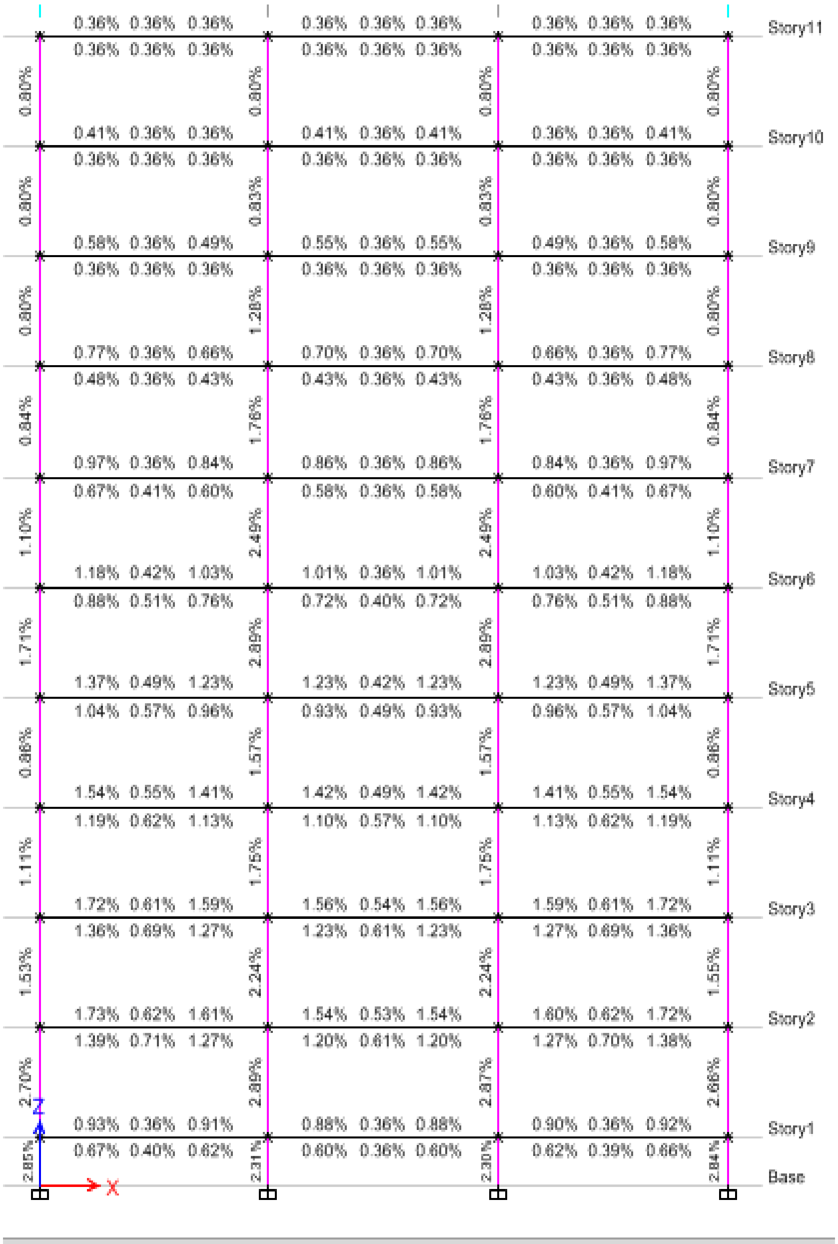


Figure No. 4.30Rebar percentage

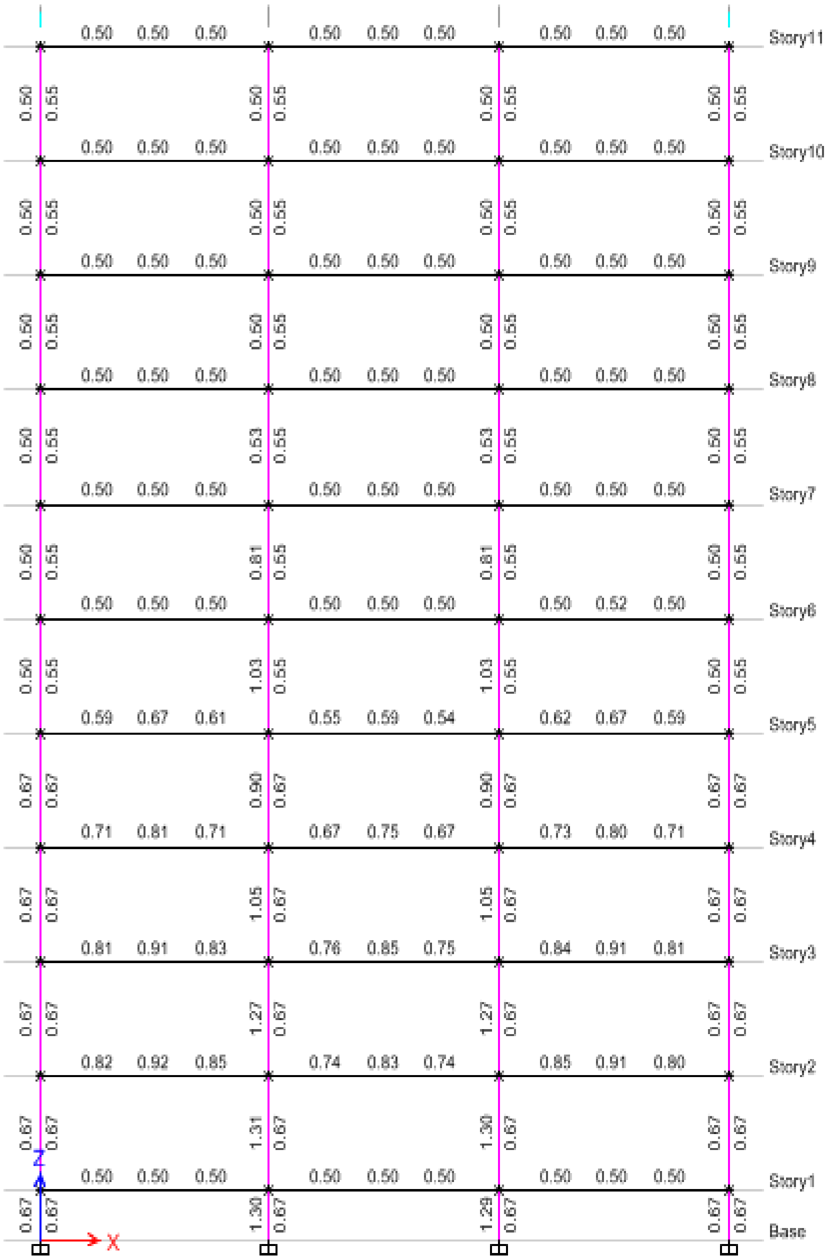


Figure No. 4.31 Shear reinforcement

FRAME 3

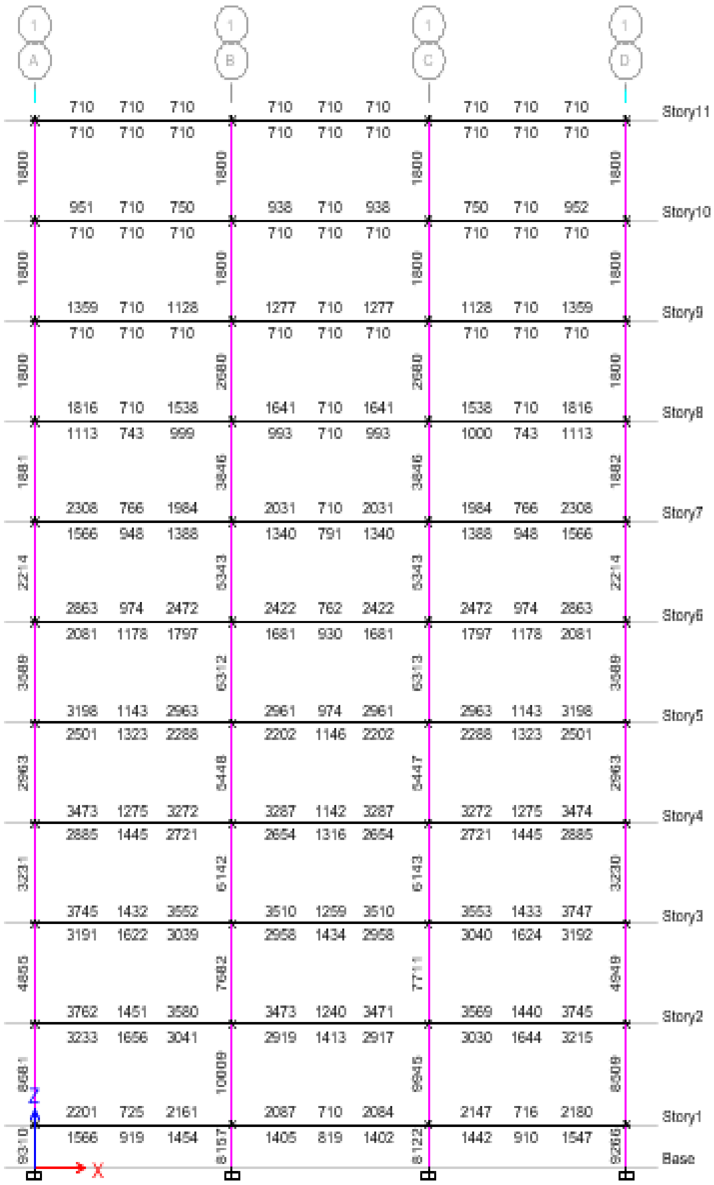


Figure No. 4.32 Longitudinal reinforcement

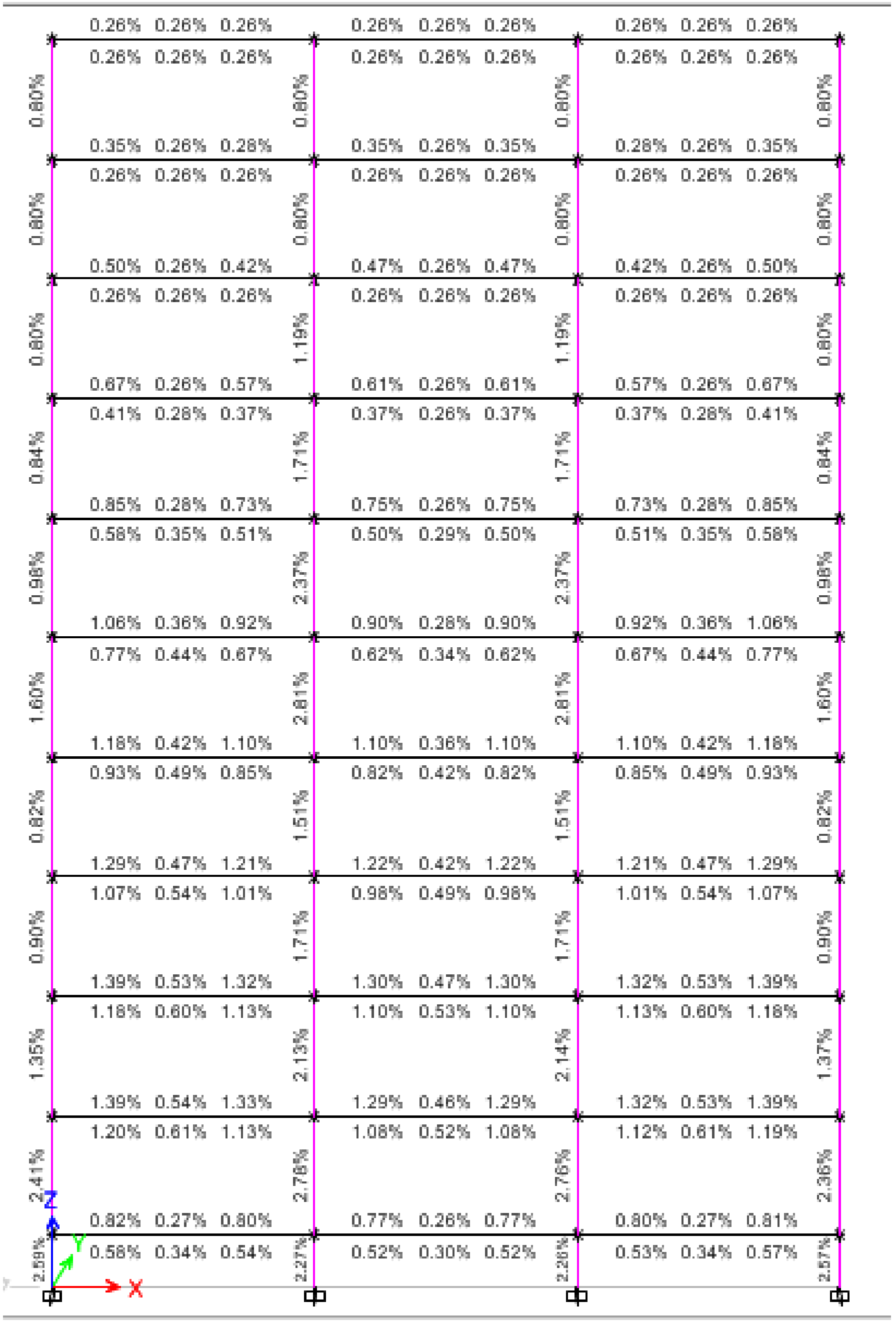


Figure No. 4.33 Rebar percentage

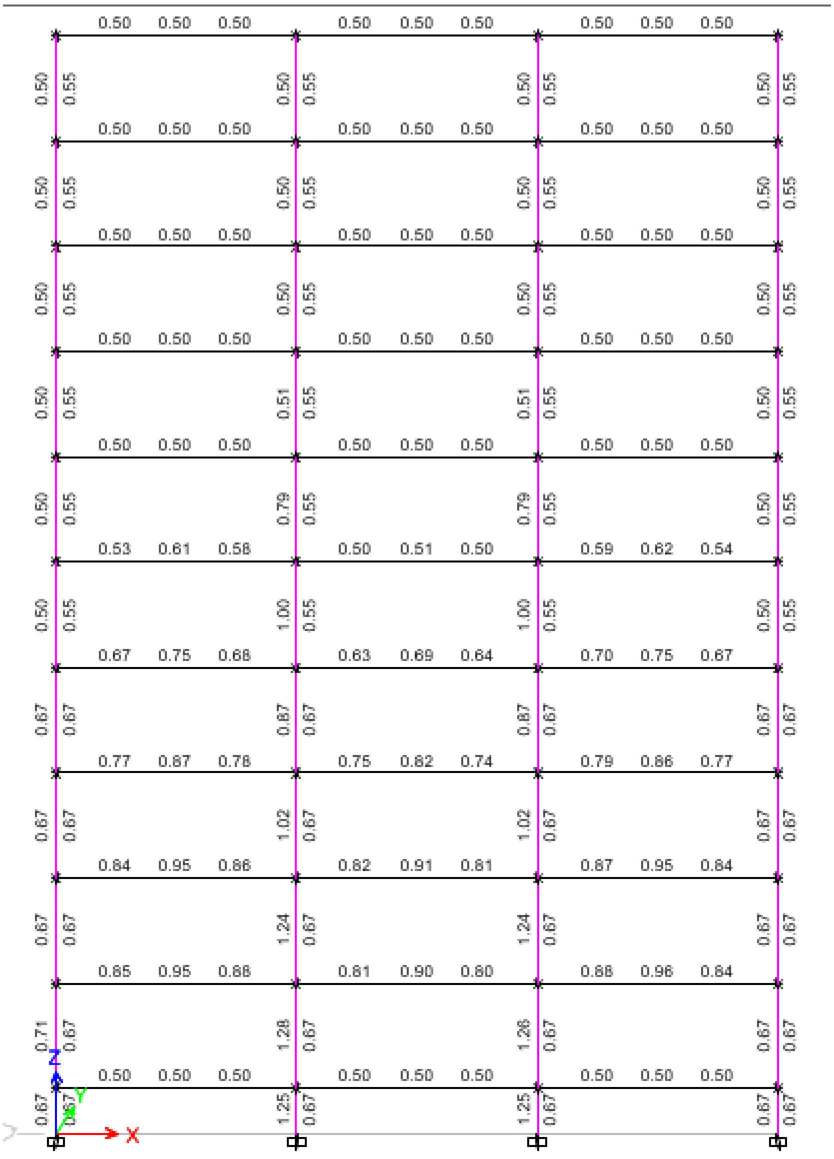


Figure No. 4.34 Shear reinforcement

Frame 4

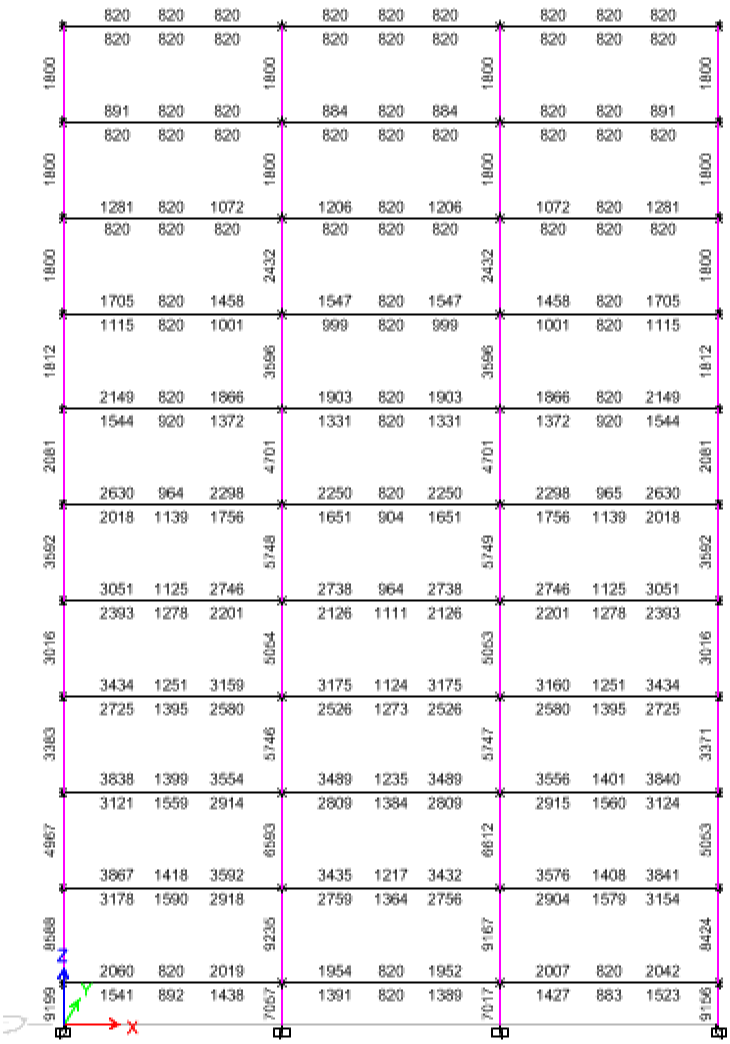


Figure No. 4.35 Longitudinal reinforcements

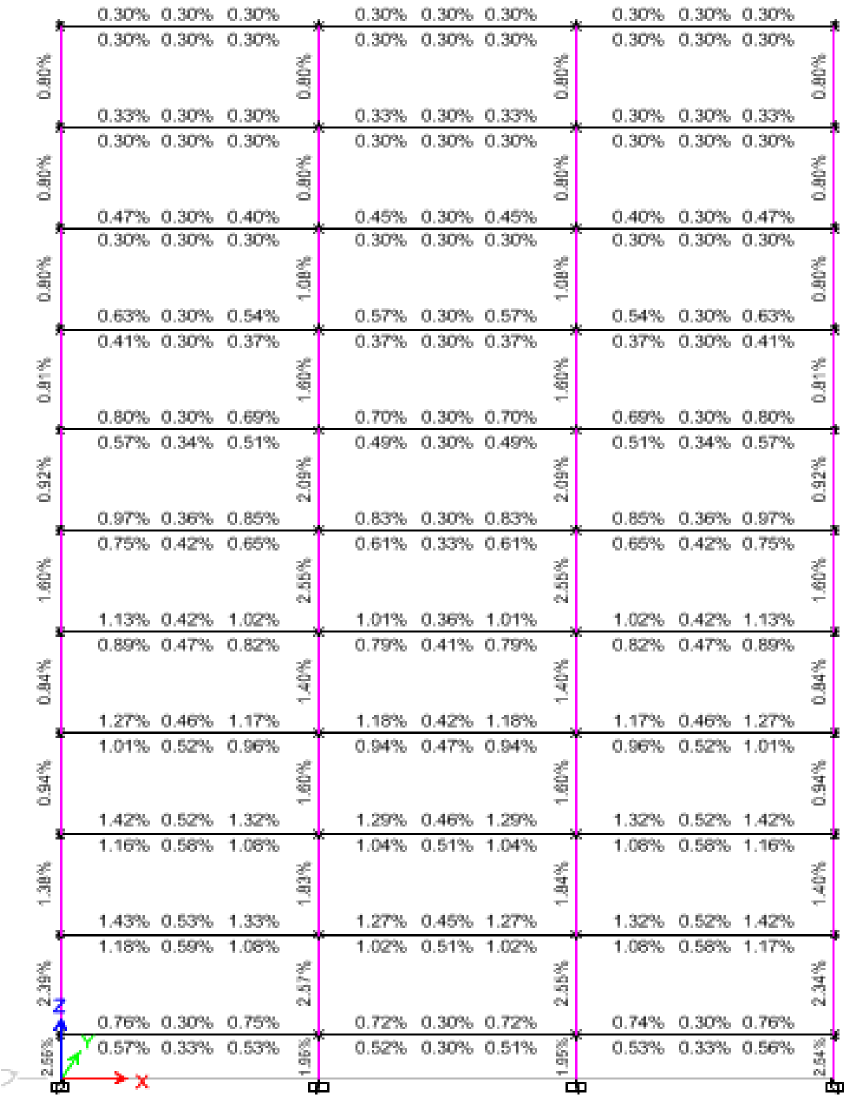


Figure 4.36 Rebar percentage

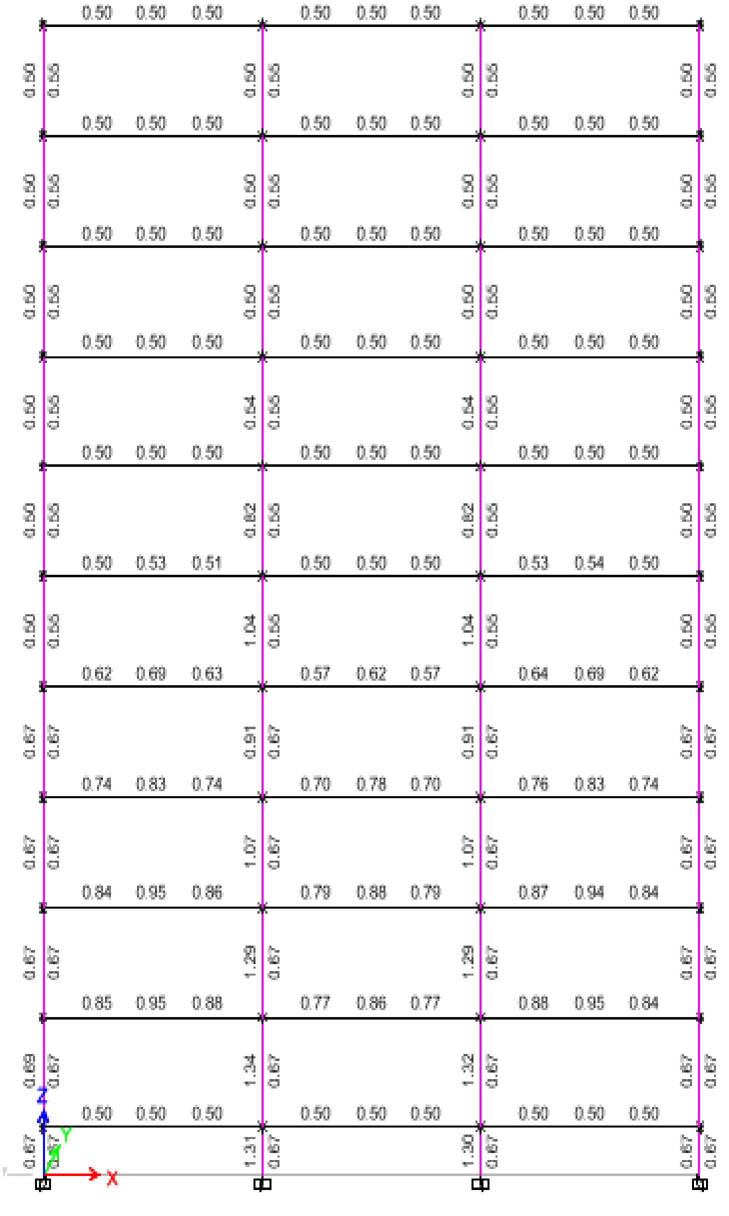


Figure 4.37 Shear reinforcement

4.5 PUSHOVER RESULTS

4.5.1 FRAME 1

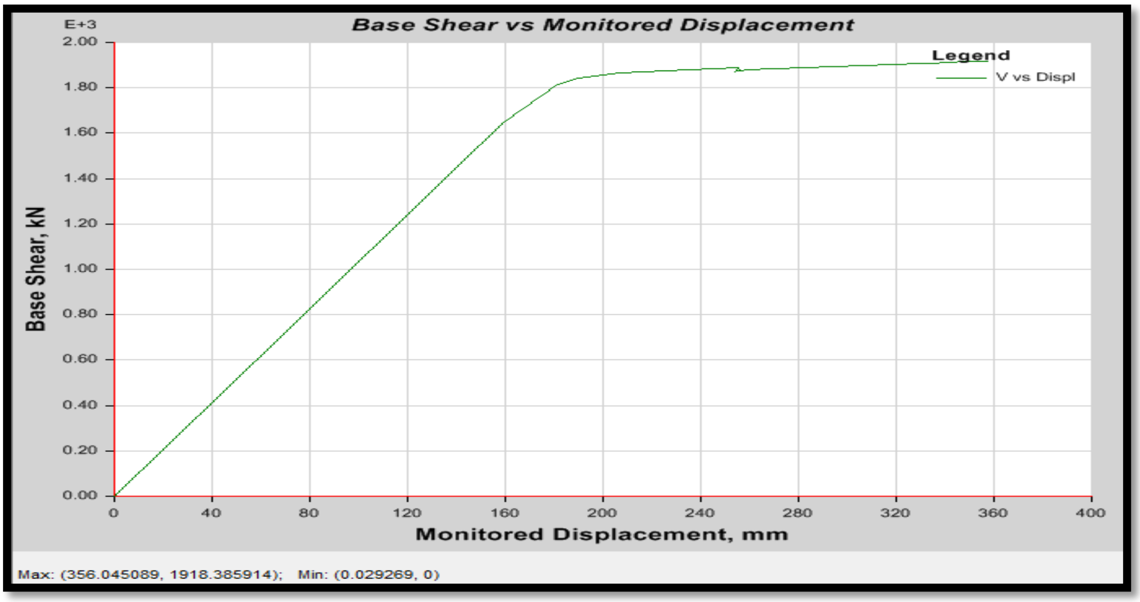


Figure 4.38 Base shear Vs Monitored displacement curve

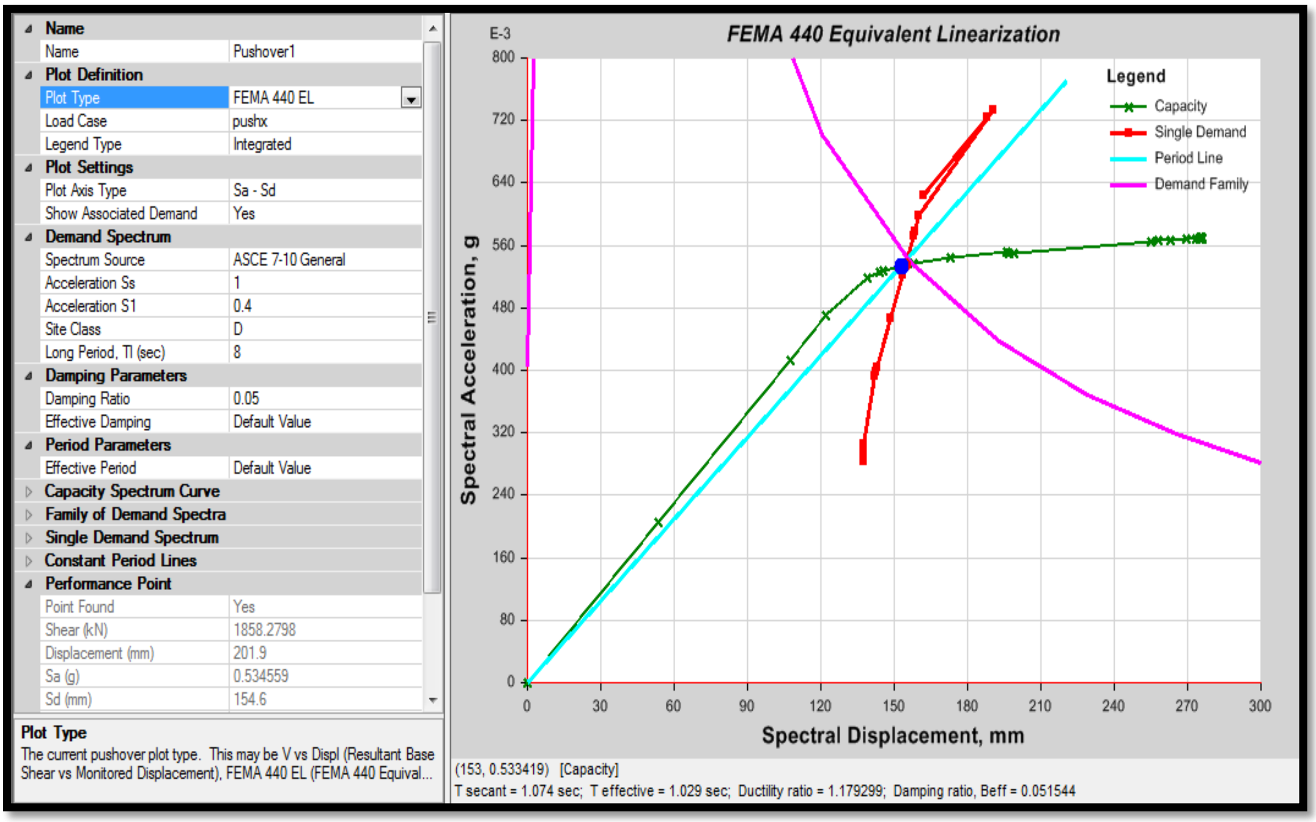


Figure No. 4.39 Spectral Accelration VS Spectral displacement curve

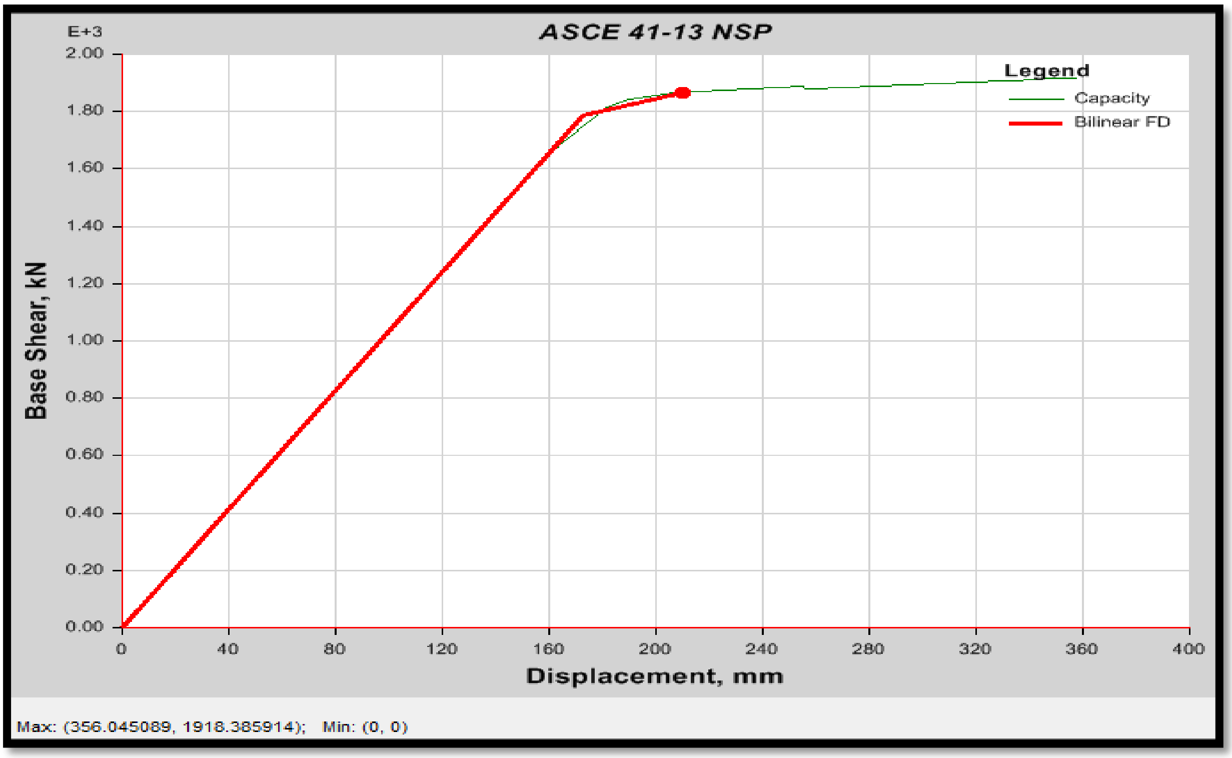


Figure No. 4.40 Bilinear capacity curve

Name	Pushover1
Plot Definition	
Plot Type	ASCE 41-13 NSP
Load Case	pushx
Legend Type	Integrated
Demand Spectrum	
Damping Ratio	0.05
Spectrum Source	ASCE 7-10 General
Acceleration Ss	1
Acceleration S1	0.4
Site Class	D
Long Period, Tl (sec)	8
Include SSI	No
C2 Type	Default Value
Cm Type	Default Value
Capacity Curve	
Visible	Yes
Line Type	Solid
Line Width	1 Pixel (Regular)
Line Color	Green

Figure No. 4.41 Demand spectrum data

Bilinear Force-Displacement Curve	
Visible	Yes
Line Type	Solid
Line Width	3 Pixels
Line Color	Red
Target Displacement Results	
Displ. (mm)	209.7
Shear (kN)	1865.7306
Calculated Parameters	
C0	1.305661
C1	1
C2	1
Sa, g	0.624329
Te (sec)	1.023
Ki (kN/mm)	10.335
Ke (kN/mm)	10.335
Ti (sec)	1.023
Alpha	0.210338
uStrength	1.635136
Dy (mm)	172.8
Vy (kN)	1785.4787
Weight (kN)	4676.2176
Cm	1

Figure No. 4.42 Target displacement results

Frame 2

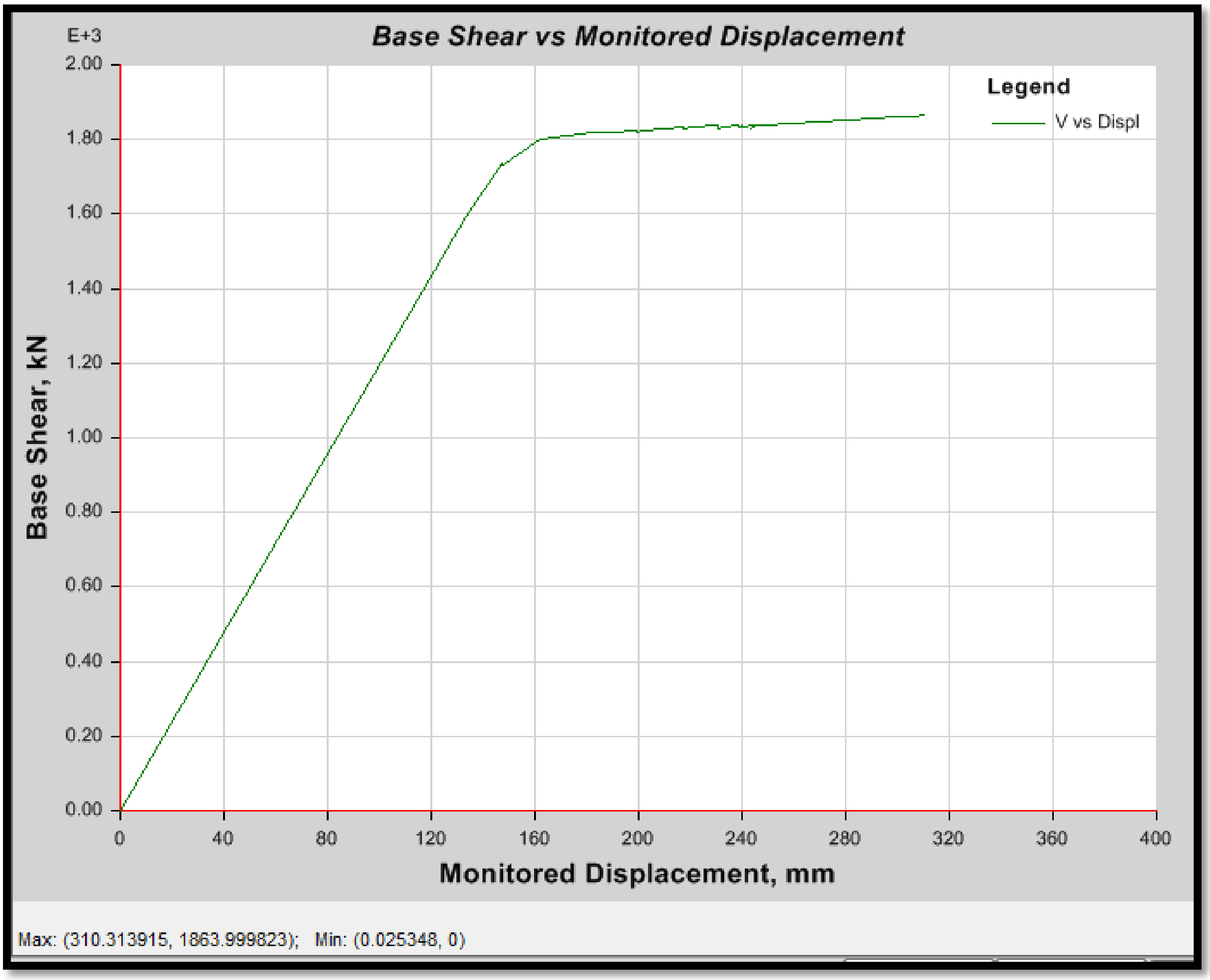


Figure No. 4.43 Base shear VS Monitored displacement curve

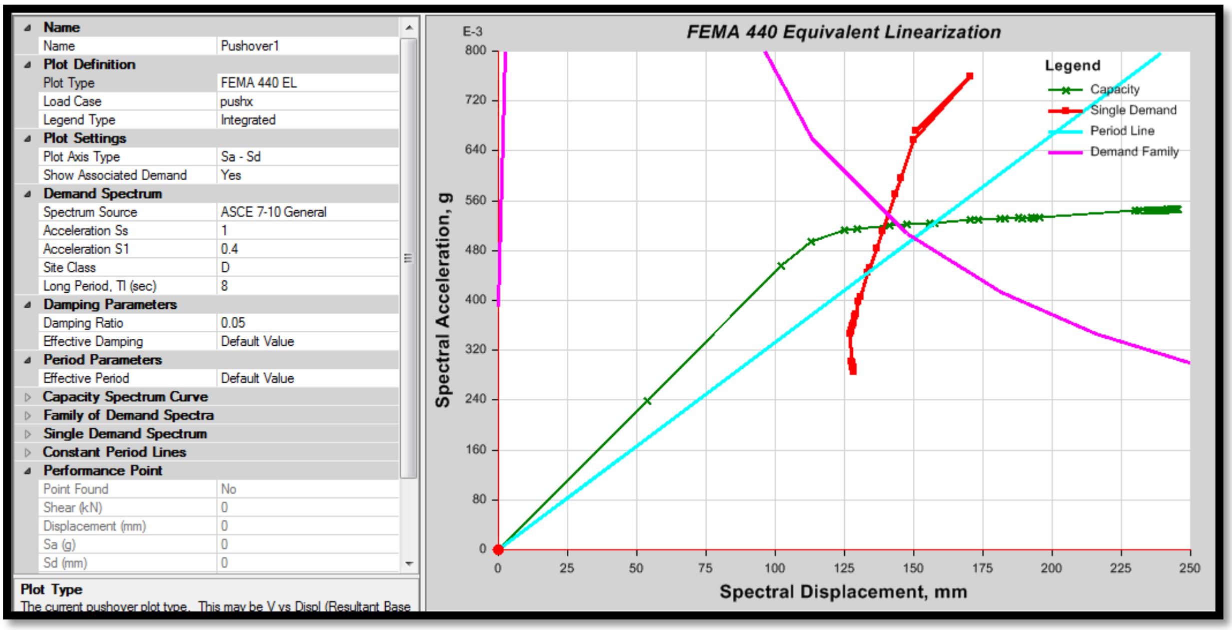


Figure 4.44 Spectral acceleration VS Spectral displacement graph

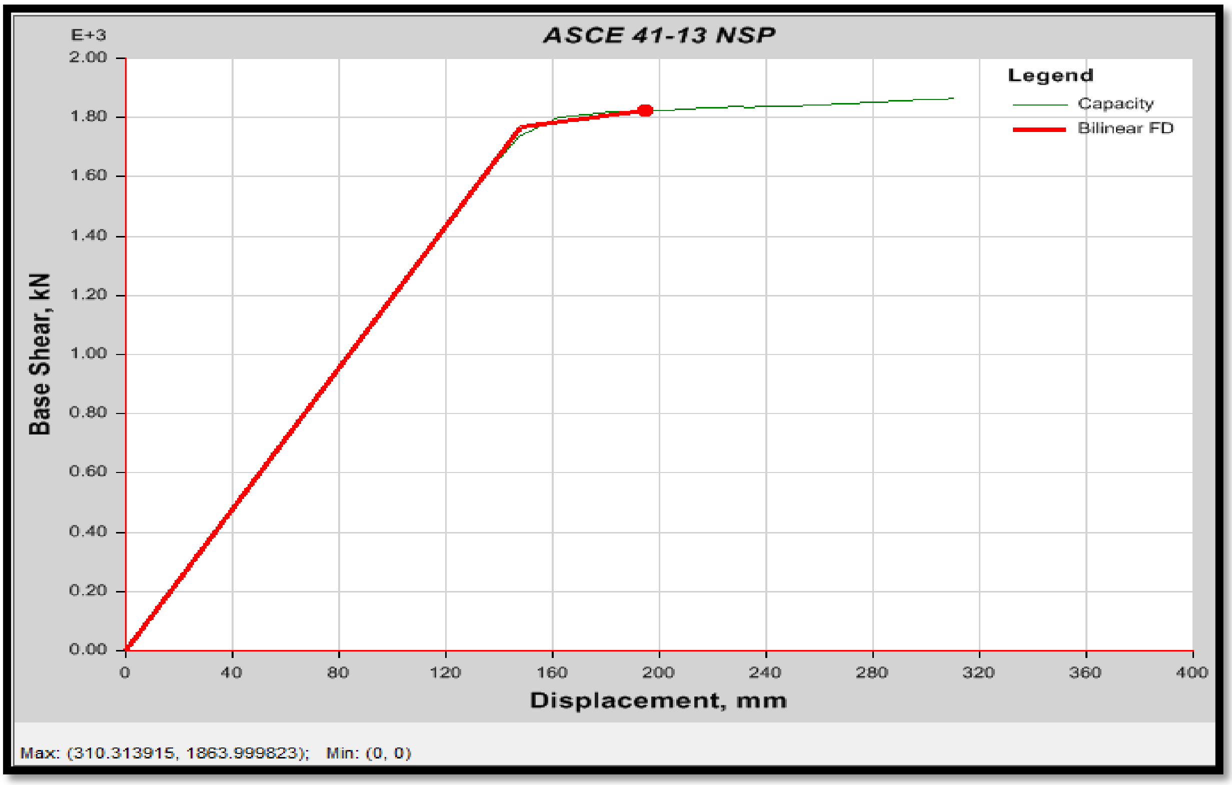


Figure 4.45 Bilinear capacity curves

Name	
Name	Pushover2
Plot Definition	
Plot Type	ASCE 41-13 NSP
Load Case	pushx
Legend Type	Integrated
Demand Spectrum	
Damping Ratio	0.05
Spectrum Source	ASCE 7-10 General
Acceleration Ss	1
Acceleration S1	0.4
Site Class	D
Long Period, Tl (sec)	8
Include SSI	No
C2 Type	Default Value
Cm Type	Default Value
Capacity Curve	
Visible	Yes
Line Type	Solid
Line Width	1 Pixel (Regular)
Line Color	Green

Figure 4.46 Demand spectrum data

Bilinear Force-Displacement Curve	
Visible	Yes
Line Type	Solid
Line Width	3 Pixels
Line Color	Red
Target Displacement Results	
Displ. (mm)	194.5
Shear (kN)	1822.7233
Calculated Parameters	
C0	1.28198
C1	1.01428
C2	1
Sa, g	0.67133
Te (sec)	0.951
Ki (kN/mm)	11.949
Ke (kN/mm)	11.949
Ti (sec)	0.951
Alpha	0.097496
uStrength	1.775012
Dy (mm)	148
Vy (kN)	1768.5994
Weight (kN)	4676.2176
Cm	1

Figure No. 4.47 Target displacement results

FRAME 3

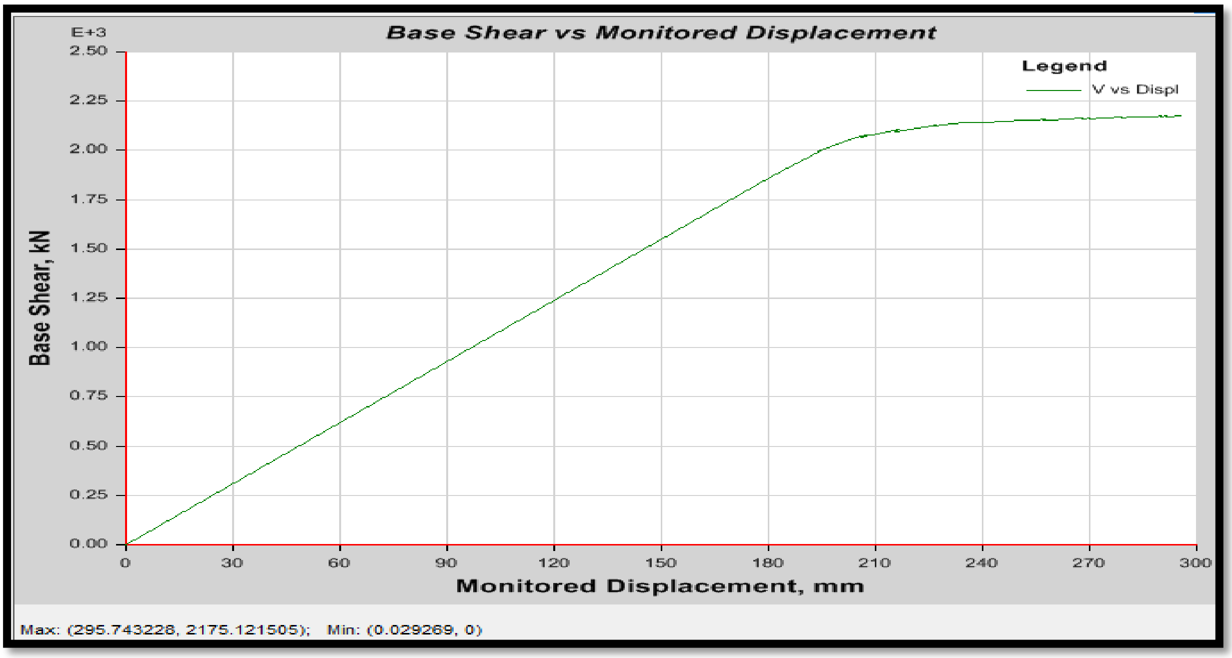


Figure No. 4.48 Base shear VS Monitored displacement curve

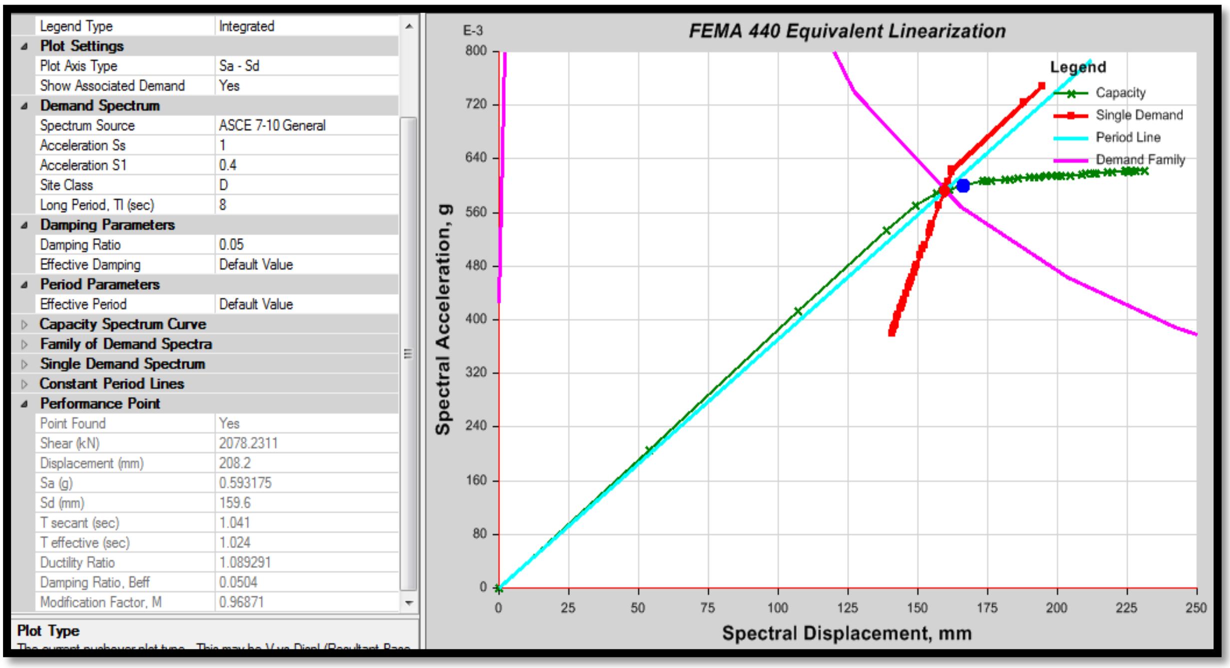


Figure No. 4.49 Spectral acceleration VS Spectral displacement curves

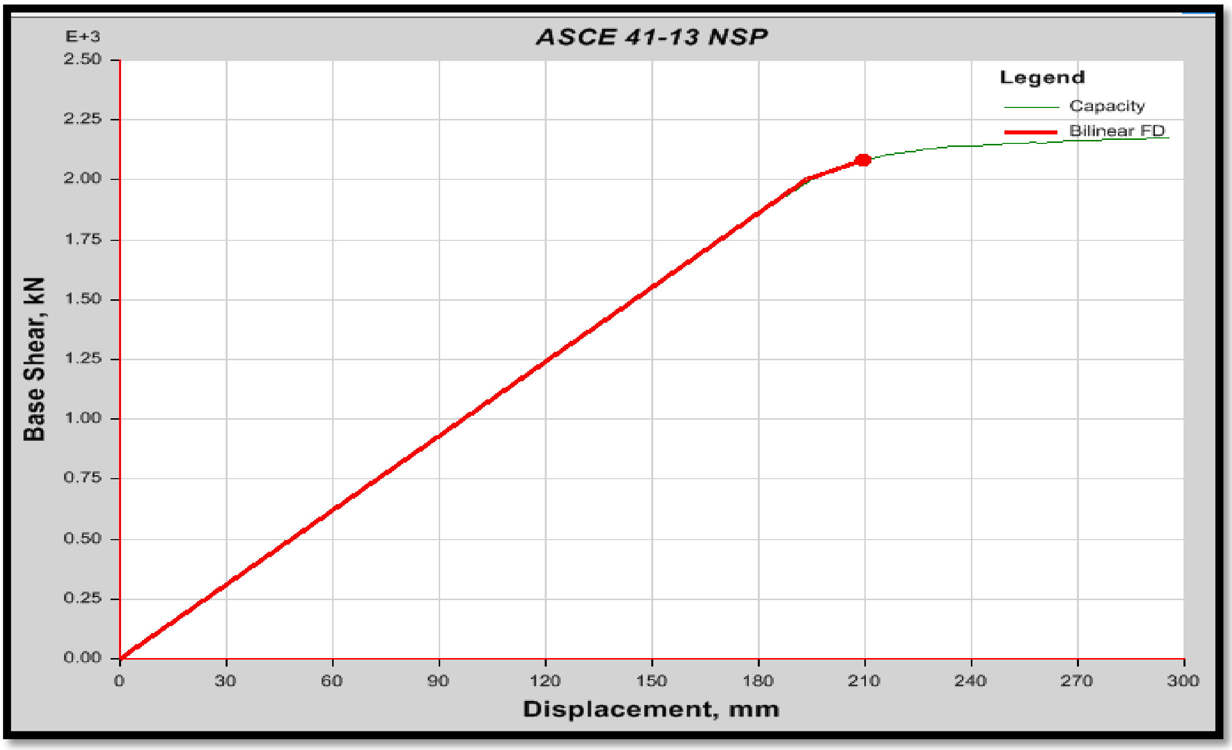


Figure No. 4.50 Bilinear capacity curve

Name	
Name	Pushover3
Plot Definition	
Plot Type	ASCE 41-13 NSP
Load Case	pushx
Legend Type	Integrated
Demand Spectrum	
Damping Ratio	0.05
Spectrum Source	ASCE 7-10 General
Acceleration Ss	1
Acceleration S1	0.4
Site Class	D
Long Period, Tl (sec)	8
Include SSI	No
C2 Type	Default Value
Cm Type	Default Value
Capacity Curve	
Visible	Yes
Line Type	Solid
Line Width	1 Pixel (Regular)
Line Color	Green

Bilinear Force-Displacement Curve	
Visible	Yes
Line Type	Solid
Line Width	3 Pixels
Line Color	Red
Target Displacement Results	
Displ. (mm)	209.5
Shear (kN)	2082.0159
Calculated Parameters	
C0	1.304151
C1	1
C2	1
Sa, g	0.624329
Te (sec)	1.023
Ki (kN/mm)	10.335
Ke (kN/mm)	10.335
Ti (sec)	1.023
Alpha	0.4977
uStrength	1.460157
Dy (mm)	193.5
Vy (kN)	1999.4418
Weight (kN)	4676.2176
Cm	1

Figure No. 4.51 Demand spectrum data

Figure No. 4.52 Target displacement results

Frame 4

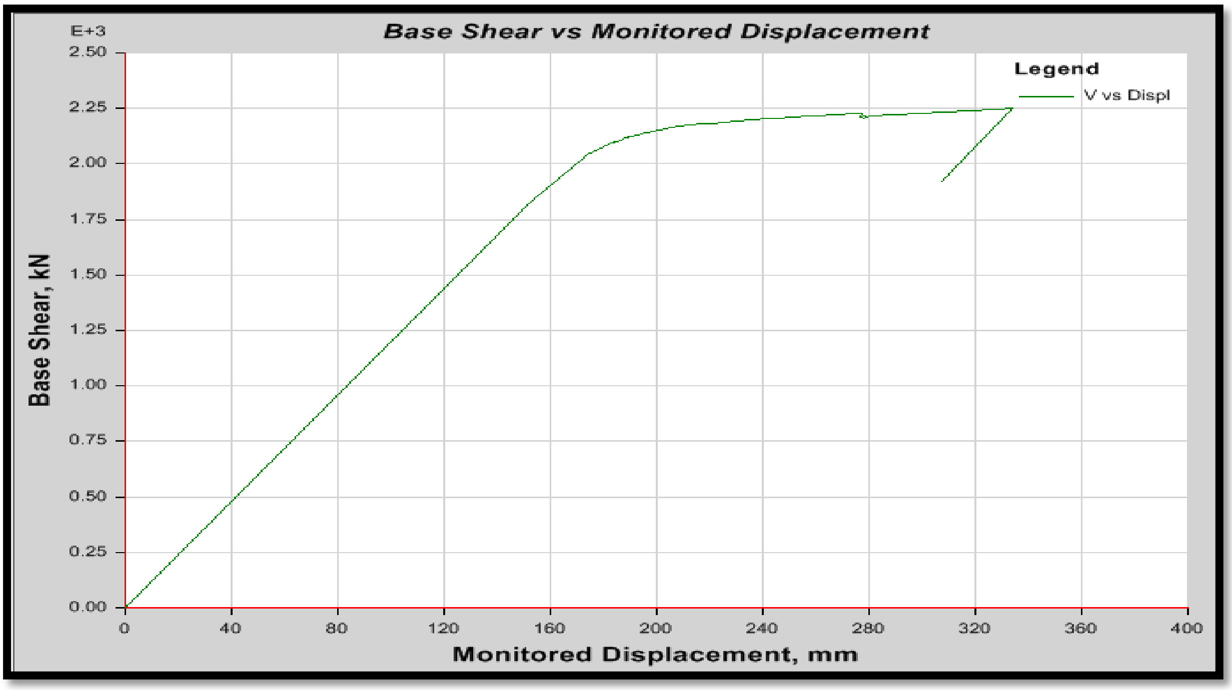


Figure No. 4.53 Base shear VS monitored displacement graph

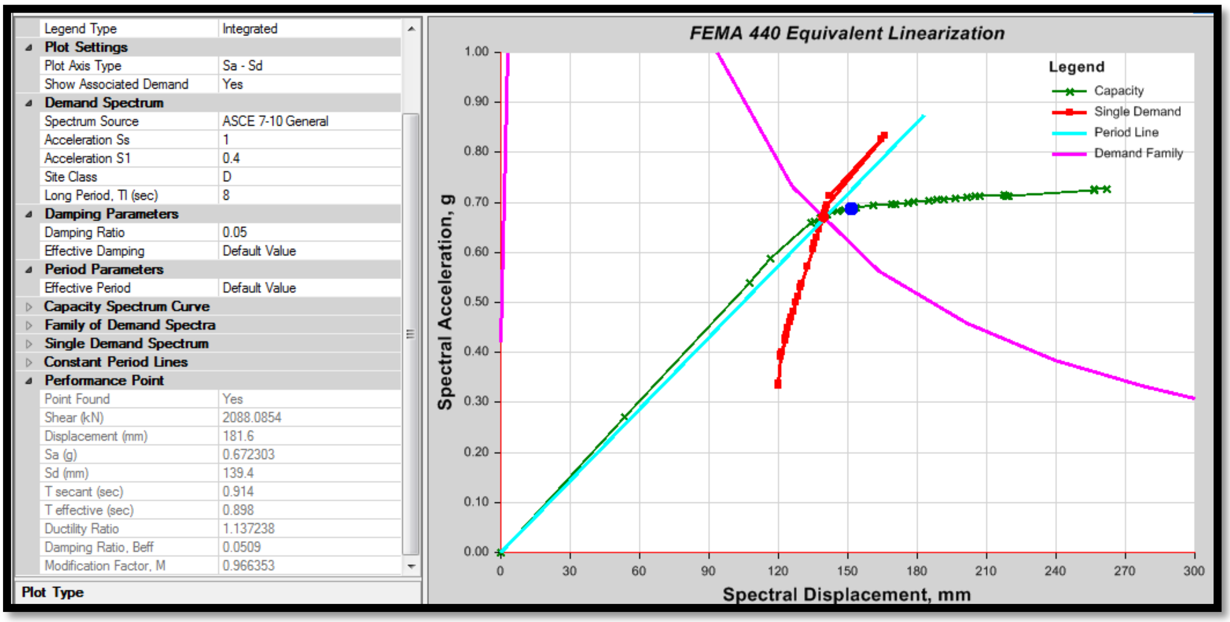


Figure No. 4.54 Spectral acceleration VS Spectral displacement graph

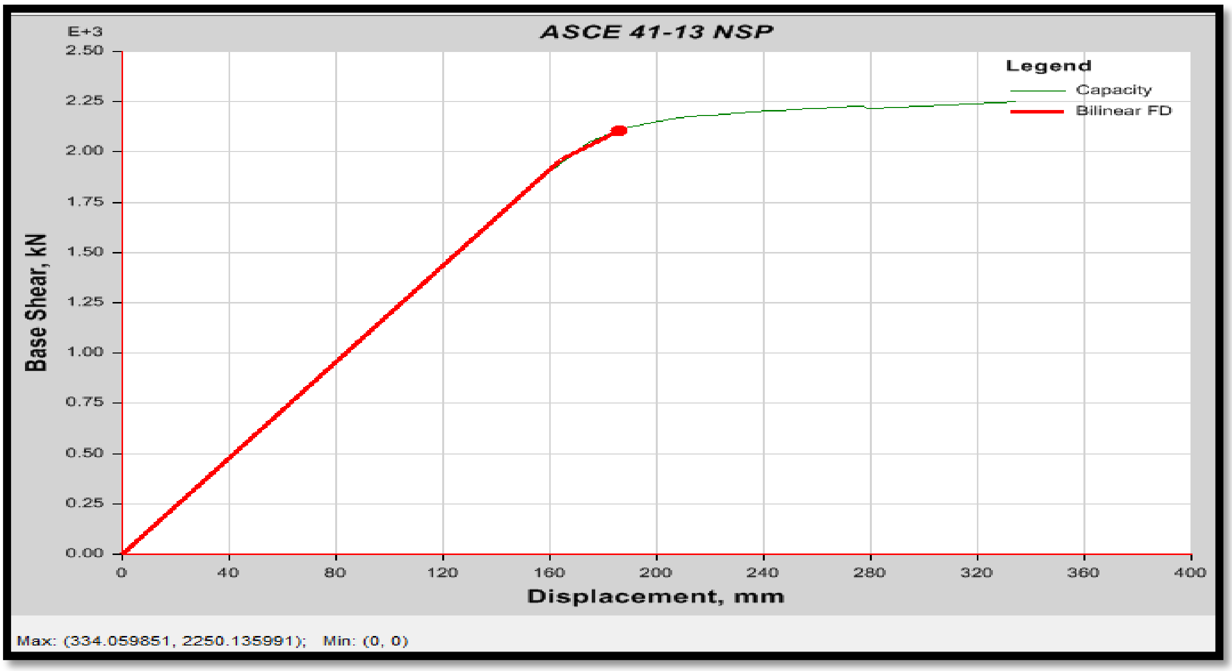


Figure No. 4.55 Bilinear capacity curve

Name	
Name	Pushover4
Plot Definition	
Plot Type	ASCE 41-13 NSP
Load Case	pushx
Legend Type	Integrated
Demand Spectrum	
Damping Ratio	0.05
Spectrum Source	ASCE 7-10 General
Acceleration Ss	1
Acceleration S1	0.4
Site Class	D
Long Period, Tl (sec)	8
Include SSI	No
C2 Type	Default Value
Cm Type	Default Value
Capacity Curve	
Visible	Yes
Line Type	Solid
Line Width	1 Pixel (Regular)
Line Color	Green

Figure No. 4.56 Demand spectrum data

Bilinear Force-Displacement Curve	
Visible	Yes
Line Type	Solid
Line Width	3 Pixels
Line Color	Red
Target Displacement Results	
Displ. (mm)	185.8
Shear (kN)	2107.0042
Calculated Parameters	
C0	1.301928
C1	1.010547
C2	1
Sa, g	0.713584
Te (sec)	0.895
Ki (kN/mm)	11.965
Ke (kN/mm)	11.965
Ti (sec)	0.895
Alpha	0.559353
uStrength	1.50663
Dy (mm)	163.7
Vy (kN)	1959.1926
Weight (kN)	4136.5524
Cm	1

Figure No. 4.57 Target displacement results

4.6 Displacements push x results

Frame 1

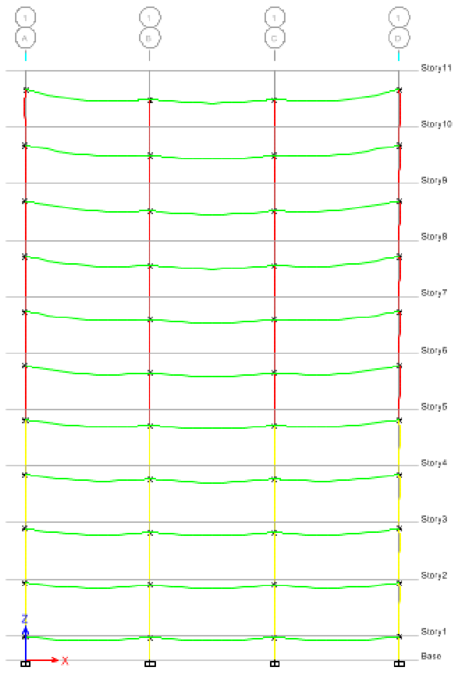


Figure 4.58 Step 0

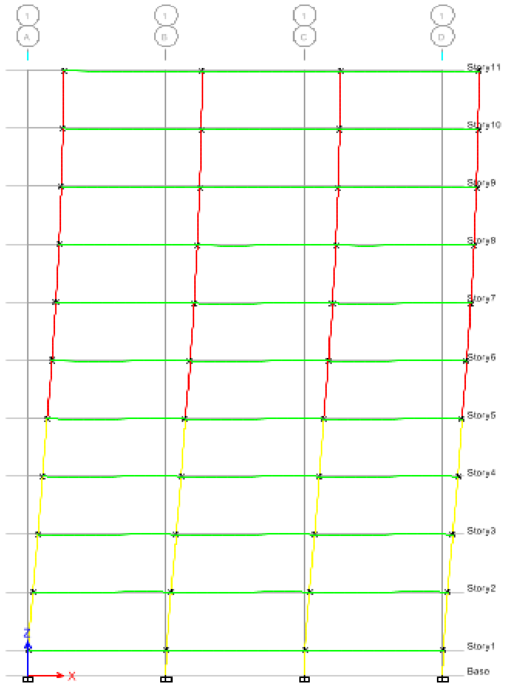


Figure 4.59 Step 1

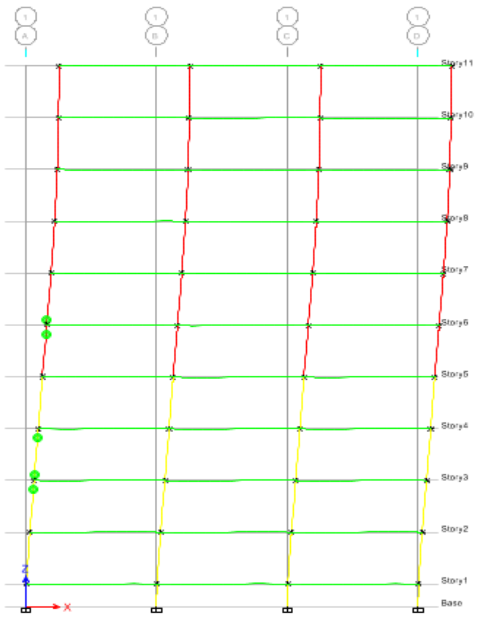


Figure 4.60 Step 9

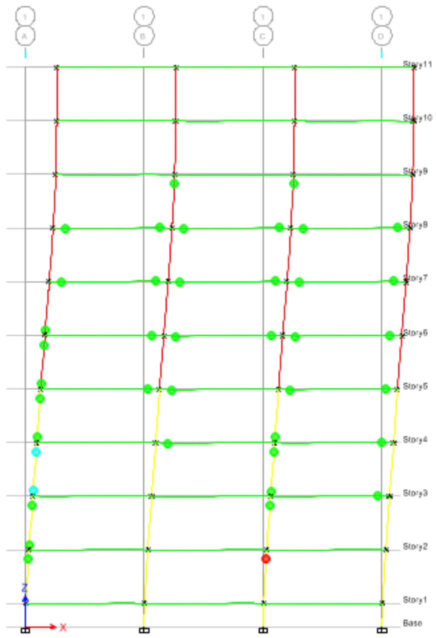


Figure 4.61 Step 94

Frame 2 Displacements push x results

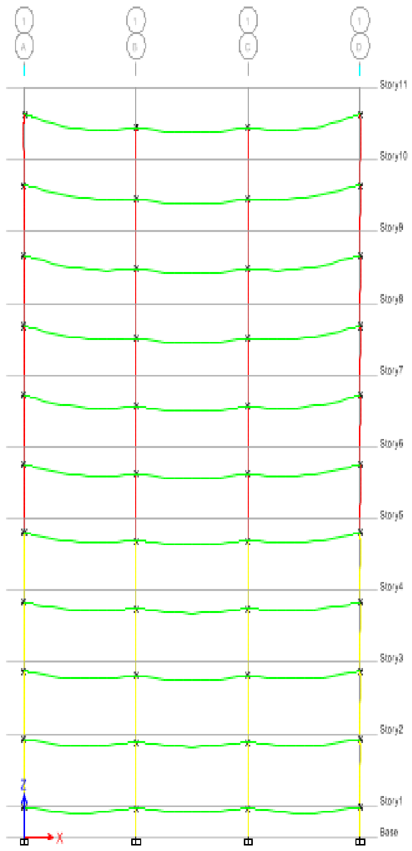


Figure 4.62 Step 0

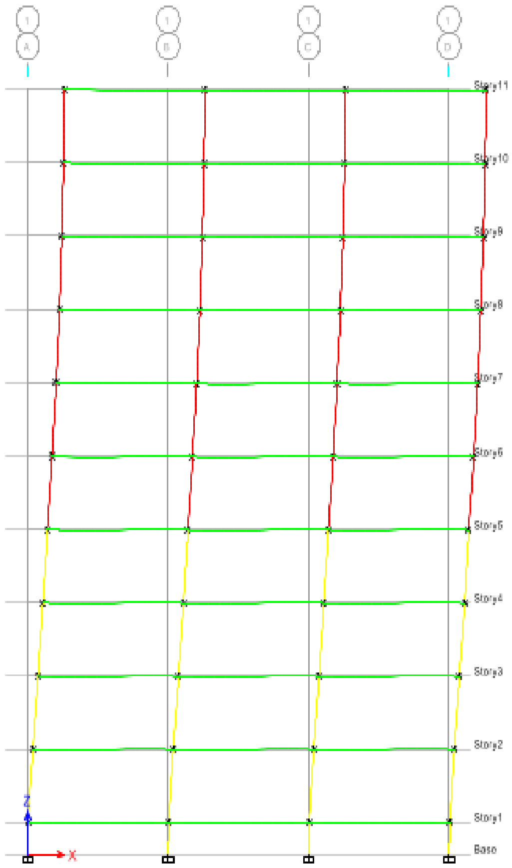


Figure 4.63 Step 1

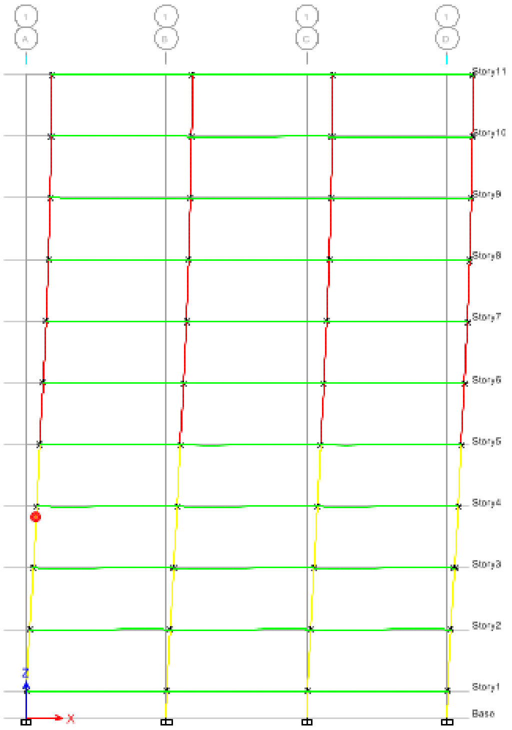


Figure 4.64Step 3

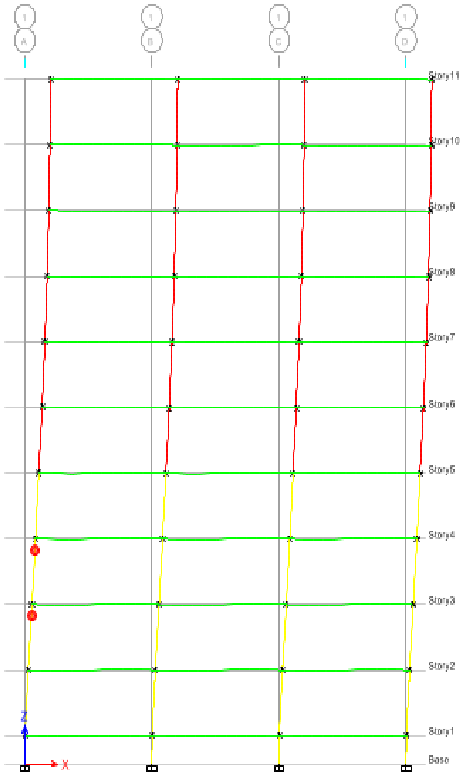


Figure 4.65 step 6

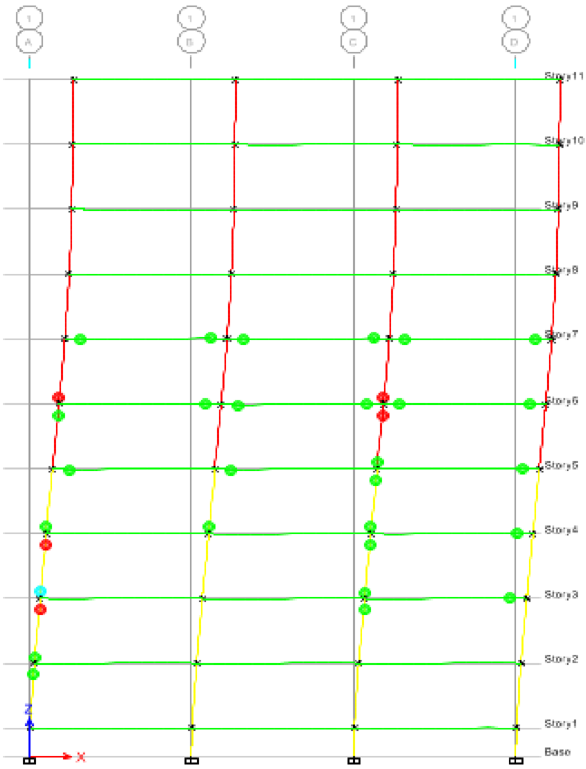


Figure 4.66 Step 92

Frame 3

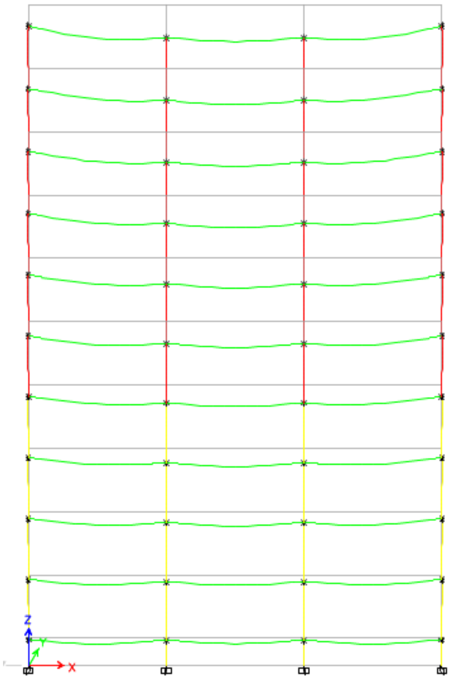


Figure 4.67 Step 0

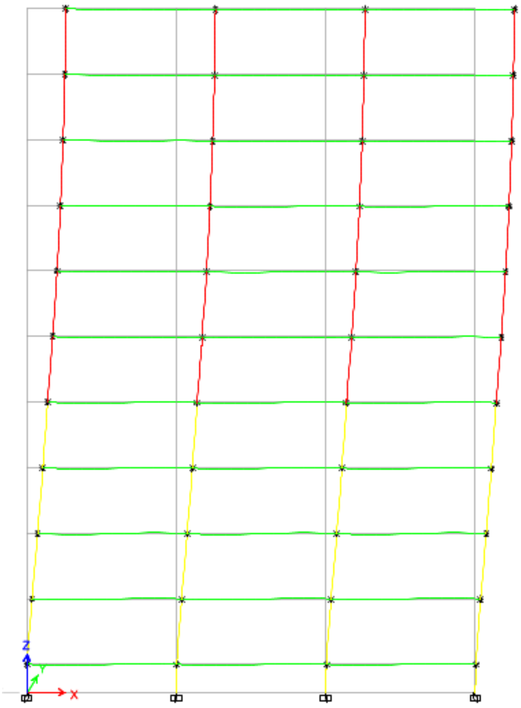


Figure 4.68 Step 1

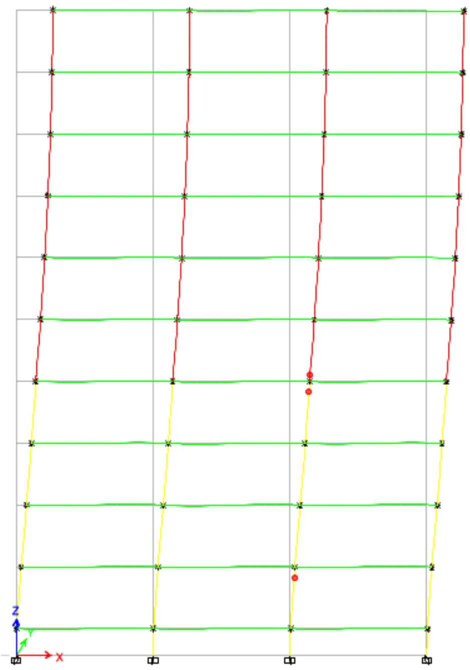


Figure No. 4.69 Step 13

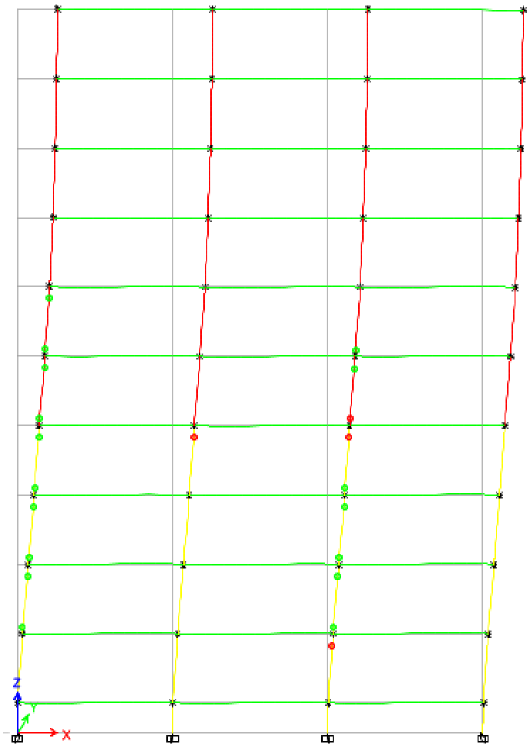


Figure No. 4.70 Step 93

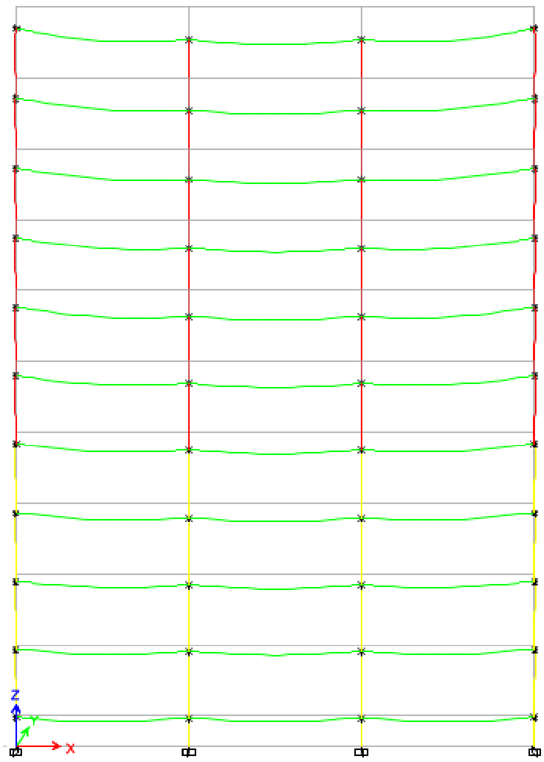


Figure No. 4.71 Step 0

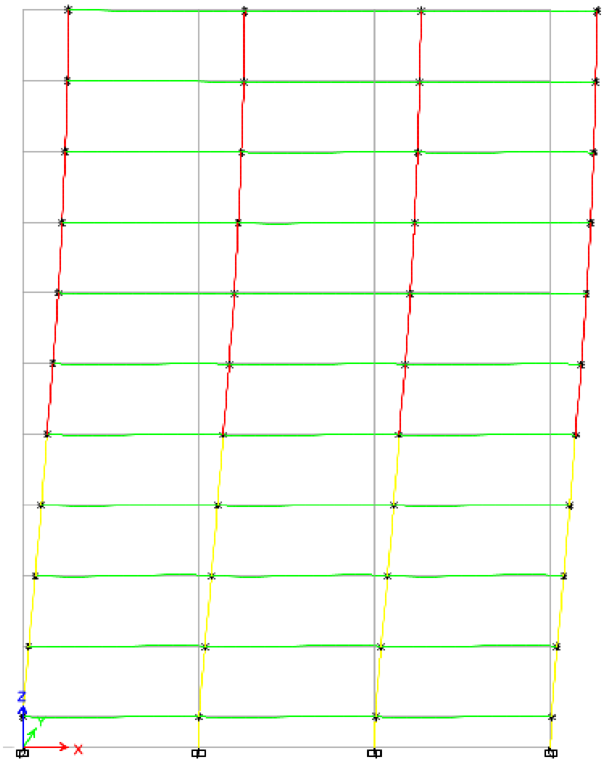


Figure No. 4.72 Step 1

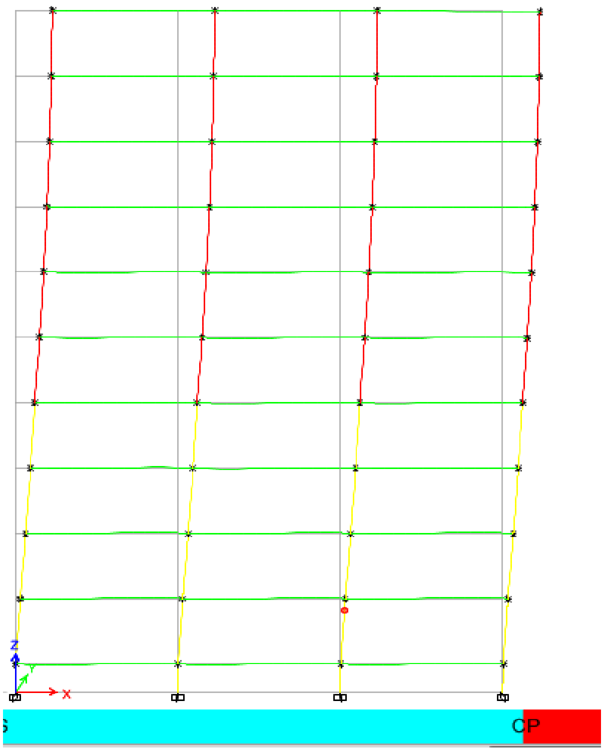


Figure No. 4.73 Step 11

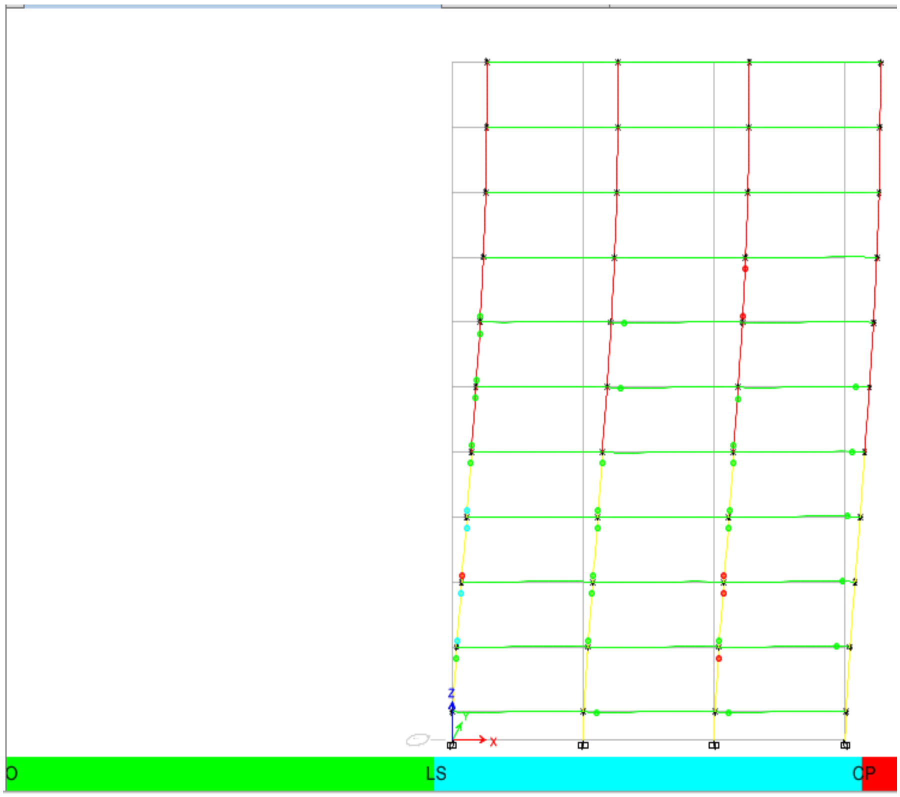


Figure No. 4.74 Step 49

CHAPTER 5

CONCLUSIONSAND DISCUSSIONS

5.1 SUMMARY

The behaviour of a multi-storey framed building during strong earthquake motions depends on the distribution of mass, stiffness, and strength in both the horizontal and vertical planes of a building. In multi-storeyed framed buildings, damage from earthquake ground motion generally initiates at locations of structural weaknesses present in the lateral load resisting frames. Further, these weaknesses tend to accentuate and concentrate the structural damage through plastification that eventually leads to complete collapse. For some cases low levels of ductility is the cause for damage. The material used in construction affects ductility of the building in one of the ways. Structural engineers have developed confidence in the design of buildings in which the distributions of mass, stiffness and strength are more or less uniform. But there is a less confidence about the ductile design of structures. Many investigations have been performed to understand the ductile behaviour of structures. It may not be possible to evaluate the seismic performance and ductility of buildings accurately using conventional static analysis as stated in IS codes of India. Therefore there arises a need for better method of nonlinear analysis such as pushover analysis outlined in FEMA 356 (2000) and ATC 40 (1996).Not much literature is available for understanding the ductile behaviour of buildings using different materials and hence this thesis is focussed on comparing ductility of frames using different grades of concrete and steel. To get a clear idea for estimating ductility of frames a detailed literature review is carried in two major areas 1. The role of ductility in seismic performance and factors affecting it 2. The use of pushover analysis tool to get ductility of frames.

To achieve the objective of the study altogether four building frames were selected for the study which are plane and orthogonal with storey heights and bay widths. Different building materials were taken for the study. Beam and column elements in this study were modelled with flexure (M3 for beams and P-M2-M3 for columns) hinges at possible plastic regions under lateral load (*i.e.*, both ends of the beams and columns). Properties of flexure hinges must simulate the actual response of reinforced concrete components subjected to lateral load. All the 4 building models with different frames are analyzed for linear/nonlinear Static/dynamic behaviour using commercial software ETABS 2015

5.2 Displacement ductility

Table 5.1: ductility

Frame	**Grade of concrete**	**Grade of steel**	**Target displacement (mm)**	**Yield displacement (mm)**	**Ductility**
1	M30	Fe 415	209.7	172.8	1.21
2	M40	Fe 415	194.5	148	1.31
3	M30	Fe 500	209.5	193.5	1.08
4	M40	Fe 500	185.8	163.7	1.13

5.3 CONCLUSIONS

1. A detailed literature review conclude the importance of incorporating ductility for earthquake resistant design, the factors affecting ductility and the use of pushover analysis for studying the non linear behaviour of frames and finding the ductility.
2. The detailed analysis conclude that grades of steel and concrete used significantly affect the yield displacements and target displacements and hence ductility.
3. The higher the grade of concrete used the more ductile the frame is.
4. Grade of steel Fe500 or less should be used in order to achieve more ductile structure
5. After analytical investigation it is concluded that the frame with M40 grade concrete and fe 415 grade steel is most ductile.

5.4 Scope for future work

1. The present study is limited to two dimensional frames and earthquake lateral load in one direction only. There is future scope of taking these frames as three dimensional and earthquake lateral load in two directions.

2. In the present study the earthquake load is taken in rectangular pattern, in future inverted triangular load can be taken into account.

3. The effect of response reduction factors can also be taken into account with materials.

4. The effect of soil structure interaction can be taken into account.

5. Frames can be compared using shear walls and different materials also.

APPENDIX A
HINGE RESULTS

Table nos. A1- A4 shows the performance of frame by hinge results calculated from base shear VS monitored displacement graph using ETABS 2015 software.

FRAME 1.

Table A1: Hinge results from base shear vs monitored displacement

TABLE: Base Shear vs Monitored Displacement 1													
Step	Monitored Displ	Base Force	A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS-CP	>CP	Total
	mm	kN										
0	0.02927	0	154	0	0	0	0	154	0	0	0	154
1	70	723.73	154	0	0	0	0	154	0	0	0	154
2	140	1447.3	154	0	0	0	0	154	0	0	0	154
3	159.4	1647.9	149	5	0	0	0	154	0	0	0	154
4	181.4	1813.3	116	38	0	0	0	154	0	0	0	154
5	188.3	1839.8	100	54	0	0	0	154	0	0	0	154
6	190.3	1844.4	95	59	0	0	0	154	0	0	0	154
7	206.4	1863.7	80	74	0	0	0	154	0	0	0	154
8	225.5	1875.7	68	86	0	0	0	154	0	0	0	154
9	255.3	1887.1	65	89	0	0	0	149	5	0	0	154
10	254	1872	64	90	0	0	0	148	5	0	1	154
11	256.1	1878.2	64	90	0	0	0	148	5	0	1	154
12	255.9	1875.8	64	90	0	0	0	148	5	0	1	154
13	258.4	1880.6	63	91	0	0	0	146	7	0	1	154
14	330.1	1908	50	104	0	0	0	117	36	0	1	154
15	334.3	1909.2	50	104	0	0	0	116	37	0	1	154
16	334.3	1909.6	50	104	0	0	0	116	37	0	1	154
17	340.6	1911.3	49	105	0	0	0	114	39	0	1	154
18	340.7	1912.1	49	105	0	0	0	114	39	0	1	154
19	349	1914.6	49	105	0	0	0	113	39	1	1	154
20	349.1	1915.9	49	105	0	0	0	113	39	1	1	154
21	354	1917.2	49	105	0	0	0	113	39	1	1	154
22	354.1	1917.9	49	105	0	0	0	113	39	1	1	154
23	356	1918.4	49	105	0	0	0	113	39	1	1	154
24	356	1918	49	105	0	0	0	113	39	1	1	154
25	356.7	1918.2	49	105	0	0	0	113	39	1	1	154
26	356.8	1918.2	49	105	0	0	0	113	39	1	1	154
27	356.8	1918.2	49	105	0	0	0	113	39	1	1	154
28	356.8	1918.2	49	105	0	0	0	113	39	1	1	154
29	356.8	1918.2	49	105	0	0	0	113	39	1	1	154
30	356.8	1918.2	49	105	0	0	0	113	39	1	1	154
31	356.8	1918.2	49	105	0	0	0	113	39	1	1	154
32	356.8	1918.2	49	105	0	0	0	113	39	1	1	154

Step	Monitored Displ	Base Force	A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS-CP	>CP	Total
	mm	kN										
35	356.8	1918.2	49	105	0	0	0	113	39	1	1	154
36	356.8	1918.2	49	105	0	0	0	113	39	1	1	154
37	356.8	1918.2	49	105	0	0	0	113	39	1	1	154
38	356.8	1918.2	49	105	0	0	0	113	39	1	1	154
39	356.8	1918.2	49	105	0	0	0	113	39	1	1	154
40	356.8	1918.2	49	105	0	0	0	113	39	1	1	154
41	356.8	1918.2	49	105	0	0	0	113	39	1	1	154
42	356.8	1918.2	49	105	0	0	0	113	39	1	1	154
43	356.9	1918.2	49	105	0	0	0	113	39	1	1	154
44	356.9	1918.2	49	105	0	0	0	113	39	1	1	154
45	356.9	1918.2	49	105	0	0	0	113	39	1	1	154
46	356.9	1918.2	49	105	0	0	0	113	39	1	1	154
47	356.9	1918.2	49	105	0	0	0	113	39	1	1	154
48	356.9	1918.2	49	105	0	0	0	113	39	1	1	154
49	356.9	1918.2	49	105	0	0	0	113	39	1	1	154
50	356.9	1918.2	49	105	0	0	0	113	39	1	1	154
51	356.9	1918.2	49	105	0	0	0	113	39	1	1	154
52	356.9	1918.2	49	105	0	0	0	113	38	2	1	154
53	356.9	1918.2	49	105	0	0	0	113	38	2	1	154
54	356.9	1918.2	49	105	0	0	0	113	38	2	1	154
55	356.9	1918.2	49	105	0	0	0	113	38	2	1	154
56	356.9	1918.2	49	105	0	0	0	113	38	2	1	154
57	356.9	1918.2	49	105	0	0	0	113	38	2	1	154
58	356.9	1918.2	49	105	0	0	0	113	38	2	1	154
59	356.9	1918.2	49	105	0	0	0	113	38	2	1	154
60	356.9	1918.2	49	105	0	0	0	113	38	2	1	154
61	357	1918.2	49	105	0	0	0	113	38	2	1	154
62	357	1918.2	49	105	0	0	0	113	38	2	1	154
63	357	1918.2	49	105	0	0	0	113	38	2	1	154
64	357	1918.2	49	105	0	0	0	113	38	2	1	154
65	357	1918.2	49	105	0	0	0	113	38	2	1	154
66	357	1918.2	49	105	0	0	0	113	38	2	1	154
67	357	1918.2	49	105	0	0	0	113	38	2	1	154
68	357	1918.2	49	105	0	0	0	113	38	2	1	154
69	357	1918.2	49	105	0	0	0	113	38	2	1	154
70	357	1918.2	49	105	0	0	0	113	38	2	1	154
71	357	1918.2	49	105	0	0	0	113	38	2	1	154
72	357	1918.2	49	105	0	0	0	113	38	2	1	154
73	357	1918.2	49	105	0	0	0	113	38	2	1	154
74	357	1918.2	49	105	0	0	0	113	38	2	1	154
75	357	1918.2	49	105	0	0	0	113	38	2	1	154
76	357	1918.2	49	105	0	0	0	113	38	2	1	154
77	357.1	1918.2	49	105	0	0	0	113	38	2	1	154

Step	Monitored Displ	Base Force	A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS-CP	>CP	Total
	mm	kN										
80	357.1	1918.2	49	105	0	0	0	113	38	2	1	154
81	357.1	1918.2	49	105	0	0	0	113	38	2	1	154
82	357.1	1918.2	49	105	0	0	0	113	38	2	1	154
83	357.1	1918.2	49	105	0	0	0	113	38	2	1	154
84	357.1	1918.2	49	105	0	0	0	113	38	2	1	154
85	357.1	1918.2	49	105	0	0	0	113	38	2	1	154
86	357.1	1918.2	49	105	0	0	0	113	38	2	1	154
87	357.1	1918.2	49	105	0	0	0	113	38	2	1	154
88	357.1	1918.2	49	105	0	0	0	113	38	2	1	154
89	357.1	1918.2	49	105	0	0	0	113	38	2	1	154
90	357.1	1918.2	49	105	0	0	0	113	38	2	1	154
91	357.1	1918.2	49	105	0	0	0	113	38	2	1	154
92	357.1	1918.2	49	105	0	0	0	113	38	2	1	154
93	357.1	1918.2	49	105	0	0	0	113	38	2	1	154
94	357.1	1917.8	49	105	0	0	0	113	38	2	1	154

FRAME 2.

Table A2: Hinge results from base shear vs monitored displacement

TABLE: Base Shear vs Monitored Displacement2												
Step	Monitored Displ	Base Force	A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS-CP	>CP	Total
	mm	kN										
0	0.02535	0	154	0	0	0	0	154	0	0	0	154
1	70	836.7144	154	0	0	0	0	154	0	0	0	154
2	133.4	1594.3005	151	3	0	0	0	154	0	0	0	154
3	147.4	1735.8129	125	29	0	0	0	153	0	0	1	154
4	146.9	1729.1232	125	29	0	0	0	153	0	0	1	154
5	162.2	1798.7174	94	60	0	0	0	152	0	0	2	154
6	167.6	1806.8649	85	69	0	0	0	152	0	0	2	154
7	167.6	1806.2401	85	69	0	0	0	152	0	0	2	154
8	181.9	1817.1428	72	82	0	0	0	151	0	0	3	154
9	189.3	1819.8241	72	82	0	0	0	151	0	0	3	154
10	199.3	1825.4277	70	84	0	0	0	151	0	0	3	154
11	199	1819.9358	70	84	0	0	0	151	0	0	3	154
12	201.5	1823.7467	70	84	0	0	0	150	1	0	3	154
13	201.4	1823.1907	70	84	0	0	0	150	1	0	3	154
14	217.6	1832.1843	69	85	0	0	0	150	1	0	3	154
15	217.3	1827.3592	69	85	0	0	0	150	1	0	3	154
16	220.7	1831.6815	69	85	0	0	0	149	2	0	3	154
17	230.9	1836.5937	67	87	0	0	0	144	7	0	3	154
18	230.5	1830.6678	67	87	0	0	0	144	7	0	3	154

Step	Monitored Displ	Base Force	A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS-CP	>CP	Total
	mm	kN										
21	238.4	1833.0279	66	88	0	0	0	143	8	0	3	154
22	240.2	1836.8152	66	88	0	0	0	143	8	0	3	154
23	239.9	1833.4709	66	88	0	0	0	143	8	0	3	154
24	244.4	1837.55	65	89	0	0	0	142	9	0	3	154
25	243.5	1826.8028	65	89	0	0	0	142	9	0	3	154
26	245.3	1836.6579	65	89	0	0	0	142	9	0	3	154
27	245.3	1836.1834	65	89	0	0	0	142	9	0	3	154
28	247.7	1839.3957	64	90	0	0	0	142	9	0	3	154
29	291	1858.3231	60	94	0	0	0	124	24	1	5	154
30	291	1858.3248	60	94	0	0	0	124	24	1	5	154
31	292.5	1858.7607	60	94	0	0	0	124	24	1	5	154
32	292.5	1858.7629	60	94	0	0	0	124	24	1	5	154
33	296.4	1860.0125	59	95	0	0	0	124	24	1	5	154
34	296.4	1860.015	59	95	0	0	0	124	24	1	5	154
35	296.6	1860.1527	59	95	0	0	0	124	24	1	5	154
36	296.6	1860.1553	59	95	0	0	0	124	24	1	5	154
37	297.2	1860.3699	59	95	0	0	0	124	24	1	5	154
38	297.2	1860.3724	59	95	0	0	0	124	24	1	5	154
39	297.9	1860.593	59	95	0	0	0	124	24	1	5	154
40	297.9	1860.5955	59	95	0	0	0	124	24	1	5	154
41	298.2	1860.7055	59	95	0	0	0	124	24	1	5	154
42	298.2	1860.708	59	95	0	0	0	124	24	1	5	154
43	299	1860.957	59	95	0	0	0	123	25	1	5	154
44	299	1860.9596	59	95	0	0	0	123	25	1	5	154
45	299.3	1861.0251	59	95	0	0	0	123	25	1	5	154
46	299.4	1861.0273	59	95	0	0	0	123	25	1	5	154
47	300.6	1861.2323	59	95	0	0	0	123	25	1	5	154
48	300.7	1861.235	59	95	0	0	0	123	25	1	5	154
49	300.8	1861.2977	59	95	0	0	0	123	25	1	5	154
50	300.8	1861.3007	59	95	0	0	0	123	25	1	5	154
51	301.3	1861.4945	59	95	0	0	0	123	25	1	5	154
52	301.3	1861.4971	59	95	0	0	0	123	25	1	5	154
53	301.5	1861.5841	59	95	0	0	0	123	25	1	5	154
54	301.5	1861.5866	59	95	0	0	0	123	25	1	5	154
55	301.8	1861.6394	59	95	0	0	0	123	25	1	5	154
56	301.8	1861.6419	59	95	0	0	0	123	25	1	5	154
57	302.8	1861.9843	59	95	0	0	0	123	25	1	5	154
58	302.8	1861.9868	59	95	0	0	0	123	25	1	5	154
59	303.3	1862.1463	59	95	0	0	0	123	25	1	5	154
60	303.3	1862.1486	59	95	0	0	0	123	25	1	5	154
61	303.7	1862.2426	59	95	0	0	0	123	25	1	5	154
62	303.7	1862.2447	59	95	0	0	0	123	25	1	5	154
63	303.7	1862.2447	59	95	0	0	0	123	25	1	5	154

Step	Monitored Displ	Base Force	A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS-CP	>CP	Total
	mm	kN										
66	305.1	1862.647	59	95	0	0	0	123	25	1	5	154
67	305.1	1862.6496	59	95	0	0	0	123	25	1	5	154
68	305.4	1862.7614	59	95	0	0	0	123	25	1	5	154
69	305.4	1862.764	59	95	0	0	0	123	25	1	5	154
70	305.8	1862.8448	59	95	0	0	0	123	25	1	5	154
71	305.8	1862.8473	59	95	0	0	0	123	25	1	5	154
72	306.5	1863.029	59	95	0	0	0	123	25	1	5	154
73	306.5	1863.0316	59	95	0	0	0	123	25	1	5	154
74	306.8	1863.154	59	95	0	0	0	123	25	1	5	154
75	306.8	1863.1566	59	95	0	0	0	123	25	1	5	154
76	307.7	1863.3725	59	95	0	0	0	123	25	1	5	154
77	307.7	1863.3747	59	95	0	0	0	123	25	1	5	154
78	308.4	1863.5896	59	95	0	0	0	123	25	1	5	154
79	308.4	1863.5921	59	95	0	0	0	123	25	1	5	154
80	308.8	1863.7084	59	95	0	0	0	123	25	1	5	154
81	308.8	1863.711	59	95	0	0	0	123	25	1	5	154
82	308.9	1863.7524	59	95	0	0	0	123	25	1	5	154
83	308.9	1863.755	59	95	0	0	0	123	25	1	5	154
84	309.3	1863.8559	59	95	0	0	0	122	26	1	5	154
85	309.3	1863.8585	59	95	0	0	0	122	26	1	5	154
86	309.6	1863.9544	59	95	0	0	0	121	27	1	5	154
87	309.6	1863.957	58	96	0	0	0	121	27	1	5	154
88	309.6	1863.957	58	96	0	0	0	121	27	1	5	154
89	309.7	1863.9593	58	96	0	0	0	121	27	1	5	154
90	309.7	1863.9594	58	96	0	0	0	121	27	1	5	154
91	309.7	1863.9617	58	96	0	0	0	121	27	1	5	154
92	310.3	1863.9998	58	96	0	0	0	121	27	1	5	154

FRAME 3.

Table A3: Hinge results from base shear vs monitored displacement

TABLE: Base Shear vs Monitored Displacement3												
Step	Monitored Displ	Base Force	A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS-CP	>CP	Total
	mm	kN										
0	0.02927	0	154	0	0	0	0	154	0	0	0	154
1	70	723.7286	154	0	0	0	0	154	0	0	0	154
2	140	1447.2957	154	0	0	0	0	154	0	0	0	154
3	180.9	1869.3382	149	5	0	0	0	154	0	0	0	154
4	194.4	1998.7134	136	18	0	0	0	154	0	0	0	154
5	193.9	1993.6997	136	18	0	0	0	154	0	0	0	154
6	204.3	2064.4526	113	41	0	0	0	154	0	0	0	154
7	207.3	2078.0487	107	47	0	0	0	154	0	0	0	154

Step	Monitored Displ	Base Force	A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS-CP	>CP	Total
	mm	kN										
10	208.4	2075.8348	106	48	0	0	0	154	0	0	0	154
11	210.3	2085.4405	105	49	0	0	0	154	0	0	0	154
12	216.3	2103.7296	100	54	0	0	0	154	0	0	0	154
13	215.7	2097.4365	100	54	0	0	0	151	0	0	3	154
14	225.3	2126.681	91	63	0	0	0	150	0	0	4	154
15	227	2128.8191	90	64	0	0	0	150	0	0	4	154
16	226.5	2123.2428	90	64	0	0	0	150	0	0	4	154
17	228.7	2129.0091	90	64	0	0	0	150	0	0	4	154
18	228.6	2127.7187	90	64	0	0	0	150	0	0	4	154
19	235	2138.6778	83	71	0	0	0	150	0	0	4	154
20	237	2139.9615	83	71	0	0	0	150	0	0	4	154
21	240.4	2143.7652	80	74	0	0	0	150	0	0	4	154
22	240.4	2142.3834	80	74	0	0	0	150	0	0	4	154
23	245.5	2147.4117	77	77	0	0	0	150	0	0	4	154
24	245.5	2147.4194	77	77	0	0	0	150	0	0	4	154
25	247.4	2149.1201	76	78	0	0	0	149	1	0	4	154
26	247.4	2149.1274	76	78	0	0	0	149	1	0	4	154
27	249.6	2150.5077	76	78	0	0	0	148	2	0	4	154
28	249.6	2150.5149	76	78	0	0	0	148	2	0	4	154
29	252.7	2152.9721	76	78	0	0	0	148	2	0	4	154
30	252.7	2152.9794	76	78	0	0	0	148	2	0	4	154
31	255	2154.4219	74	80	0	0	0	148	2	0	4	154
32	257.3	2156.9584	74	80	0	0	0	147	3	0	4	154
33	257	2150.9717	74	80	0	0	0	147	3	0	4	154
34	259.2	2155.2572	74	80	0	0	0	146	4	0	4	154
35	262.8	2157.396	73	81	0	0	0	146	4	0	4	154
36	268	2162.3589	73	81	0	0	0	146	4	0	4	154
37	267.9	2160.1994	73	81	0	0	0	146	4	0	4	154
38	269.9	2162.4402	73	81	0	0	0	146	4	0	4	154
39	269.8	2161.1732	73	81	0	0	0	145	5	0	4	154
40	273.3	2164.282	73	81	0	0	0	144	6	0	4	154
41	273.2	2163.9749	73	81	0	0	0	144	6	0	4	154
42	274.3	2165.0436	73	81	0	0	0	144	6	0	4	154
43	274.3	2163.1118	73	81	0	0	0	144	6	0	4	154
44	279.2	2167.412	73	81	0	0	0	140	10	0	4	154
45	279.1	2165.278	73	81	0	0	0	140	10	0	4	154
46	281.4	2168.3623	73	81	0	0	0	137	13	0	4	154
47	281.4	2167.6837	73	81	0	0	0	137	13	0	4	154
48	287	2172.5097	72	82	0	0	0	135	15	0	4	154
49	287	2172.5162	72	82	0	0	0	135	15	0	4	154
50	287.2	2172.6683	72	82	0	0	0	135	15	0	4	154
51	287.2	2172.6744	72	82	0	0	0	135	15	0	4	154
52	287.2	2172.6756	72	82	0	0	0	135	15	0	4	154

Step	Monitored Displ	Base Force	A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS-CP	>CP	Total
	mm	kN										
55	287.2	2172.6885	72	82	0	0	0	135	15	0	4	154
56	287.2	2172.6897	72	82	0	0	0	135	15	0	4	154
57	287.2	2172.6955	72	82	0	0	0	135	15	0	4	154
58	287.2	2172.6967	72	82	0	0	0	135	15	0	4	154
59	287.2	2172.7026	72	82	0	0	0	135	15	0	4	154
60	287.2	2172.7038	72	82	0	0	0	135	15	0	4	154
61	287.2	2172.7096	72	82	0	0	0	135	15	0	4	154
62	287.2	2172.7108	72	82	0	0	0	135	15	0	4	154
63	287.2	2172.7162	72	82	0	0	0	135	15	0	4	154
64	287.2	2172.7177	72	82	0	0	0	135	15	0	4	154
65	287.2	2172.7231	72	82	0	0	0	135	15	0	4	154
66	287.2	2172.7247	72	82	0	0	0	135	15	0	4	154
67	287.2	2172.7301	72	82	0	0	0	135	15	0	4	154
68	287.2	2172.7316	72	82	0	0	0	135	15	0	4	154
69	287.2	2172.737	72	82	0	0	0	135	15	0	4	154
70	287.2	2172.7385	72	82	0	0	0	135	15	0	4	154
71	287.3	2172.7439	72	82	0	0	0	135	15	0	4	154
72	287.3	2172.7454	72	82	0	0	0	135	15	0	4	154
73	287.3	2172.7508	72	82	0	0	0	135	15	0	4	154
74	287.3	2172.7523	72	82	0	0	0	135	15	0	4	154
75	287.3	2172.7578	72	82	0	0	0	135	15	0	4	154
76	287.3	2172.7593	72	82	0	0	0	135	15	0	4	154
77	287.3	2172.7647	72	82	0	0	0	135	15	0	4	154
78	287.3	2172.7662	72	82	0	0	0	135	15	0	4	154
79	287.2	2169.92	72	82	0	0	0	135	15	0	4	154
80	287.2	2169.5152	72	82	0	0	0	135	15	0	4	154
81	288.4	2171.6449	72	82	0	0	0	135	15	0	4	154
82	290.2	2173.3137	72	82	0	0	0	135	15	0	4	154
83	290.1	2171.945	72	82	0	0	0	135	15	0	4	154
84	291.5	2173.3203	72	82	0	0	0	135	15	0	4	154
85	291.3	2171.4812	72	82	0	0	0	135	15	0	4	154
86	291.3	2171.5536	72	82	0	0	0	135	15	0	4	154
87	291.3	2171.6259	72	82	0	0	0	135	15	0	4	154
88	291.3	2171.6983	72	82	0	0	0	135	15	0	4	154
89	291.3	2171.7707	72	82	0	0	0	135	15	0	4	154
90	291.3	2171.8431	72	82	0	0	0	135	15	0	4	154
91	291.3	2171.9158	72	82	0	0	0	135	15	0	4	154
92	291.3	2171.4966	72	82	0	0	0	135	15	0	4	154
93	295.7	2175.1215	71	83	0	0	0	133	17	0	4	154

FRAME 4.

Table A3: Hinge results from base shear vs monitored displacement.

TABLE: Base Shear vs Monitored Displacement													
Step	Monitored Displ	Base Force	A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS-CP	>CP	Total
	mm	kN										
0	0.02112	0	154	0	0	0	0	154	0	0	0	154
1	70	837.8197	154	0	0	0	0	154	0	0	0	154
2	140	1675.452	154	0	0	0	0	154	0	0	0	154
3	152.2	1821.73	149	5	0	0	0	154	0	0	0	154
4	174.5	2049.488	120	34	0	0	0	154	0	0	0	154
5	173.9	2041.591	120	34	0	0	0	154	0	0	0	154
6	176.3	2059.222	117	37	0	0	0	154	0	0	0	154
7	176.1	2055.955	117	37	0	0	0	154	0	0	0	154
8	179.9	2080.202	112	42	0	0	0	154	0	0	0	154
9	179.9	2078.374	112	42	0	0	0	154	0	0	0	154
10	184	2099.148	109	45	0	0	0	154	0	0	0	154
11	183.8	2095.881	109	45	0	0	0	153	0	0	1	154
12	189.2	2121.523	102	52	0	0	0	153	0	0	1	154
13	189	2116.738	102	52	0	0	0	153	0	0	1	154
14	192.9	2131.827	99	55	0	0	0	153	0	0	1	154
15	192.9	2129.942	99	55	0	0	0	153	0	0	1	154
16	196.3	2142.342	96	58	0	0	0	153	0	0	1	154
17	196.2	2138.729	96	58	0	0	0	153	0	0	1	154
18	198.9	2148.565	91	63	0	0	0	153	0	0	1	154
19	208.2	2169.267	84	70	0	0	0	153	0	0	1	154
20	217.6	2181.458	79	75	0	0	0	149	4	0	1	154
21	217.6	2181.47	79	75	0	0	0	149	4	0	1	154
22	219.5	2184.275	78	76	0	0	0	149	4	0	1	154
23	226	2189.736	77	77	0	0	0	149	4	0	1	154
24	226	2189.744	77	77	0	0	0	148	4	0	2	154
25	229.4	2192.54	76	78	0	0	0	148	4	0	2	154
26	229.4	2192.546	75	79	0	0	0	148	4	0	2	154
27	236.9	2199.071	75	79	0	0	0	148	4	0	2	154
28	236.9	2198.532	75	79	0	0	0	148	4	0	2	154
29	241.5	2202.515	74	80	0	0	0	147	5	0	2	154
30	241.5	2202.52	74	80	0	0	0	147	5	0	2	154
31	245.3	2205.622	74	80	0	0	0	145	7	0	2	154
32	245.3	2205.628	74	80	0	0	0	145	7	0	2	154
33	251.1	2209.187	73	81	0	0	0	141	11	0	2	154
34	251.2	2209.191	73	81	0	0	0	141	11	0	2	154
35	257.6	2214.59	72	82	0	0	0	138	14	0	2	154
36	257.6	2214.595	72	82	0	0	0	138	14	0	2	154
37	262.1	2217.706	71	83	0	0	0	136	16	0	2	154

Step	Monitored Displ	Base Force	A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS-CP	>CP	Total
	mm	kN										
40	277.5	2226.083	65	89	0	0	0	135	17	0	2	154
41	276.4	2210.294	65	89	0	0	0	135	17	0	2	154
42	278.9	2218.106	65	89	0	0	0	135	17	0	2	154
43	278	2206.442	65	89	0	0	0	135	17	0	2	154
44	280.1	2213.977	65	89	0	0	0	134	18	0	2	154
45	327	2245.873	60	94	0	0	0	118	27	4	5	154
46	327.1	2246.618	60	94	0	0	0	118	27	4	5	154
47	327.1	2246.621	60	94	0	0	0	118	27	4	5	154
48	334.1	2250.136	59	94	1	0	0	116	28	5	5	154
49	307.2	1923.191	59	94	0	1	0	116	28	4	6	154

APPENDIX B
BASE SHEAR AND DISPLACEMENT VALUES

Table nos. B1-B4 shows base shear and displacement values as per ASCE 41-13 using ETABS 2015.

FRAME 1

Table B1: Displacement and base shear values

TABLE: ASCE 41-13 NSP	
Displacement	**Base Shear**
mm	kN
0	0
70	723.7286
140	1447.2957
159.4	1647.9094
181.4	1813.3192
188.3	1839.8026
190.3	1844.4491
206.4	1863.6647
225.5	1875.6528
255.3	1887.0661
256.1	1878.1656
258.4	1880.627
330.1	1908.0323
334.3	1909.1954
334.3	1909.6075
340.6	1911.2521
340.7	1912.1191
349	1914.5748
349.1	1915.8866
354	1917.2068
354.1	1917.9067
356	1918.3859
356	1917.9812
356.7	1918.1995
356.8	1918.1994
356.8	1918.2009
356.8	1918.2008
356.8	1918.2023
356.8	1918.2022
356.8	1918.2037
356.8	1918.2036
356.8	1918.2051
356.8	1918.205
356.8	1918.2065

Displacement	Base Shear
mm	kN
356.8	1918.2078
356.8	1918.2093
356.8	1918.2093
356.8	1918.2107
356.8	1918.2106
356.9	1918.2121
356.9	1918.2121
356.9	1918.2135
356.9	1918.2135
356.9	1918.2149
356.9	1918.2149
356.9	1918.2163
356.9	1918.2163
356.9	1918.2177
356.9	1918.2177
356.9	1918.2191
356.9	1918.2191
356.9	1918.2205
356.9	1918.2205
356.9	1918.2219
356.9	1918.2219
356.9	1918.2233
356.9	1918.2234
357	1918.2248
357	1918.2247
357	1918.2262
357	1918.2262
357	1918.2276
357	1918.2276
357	1918.229
357	1918.229
357	1918.2304
357	1918.2304
357	1918.2318
357	1918.2319
357	1918.2333
357	1918.2333
357	1918.2347
357	1918.1653
357.1	1918.1696
357.1	1918.1703
357.1	1918.171
357.1	1918.1717
357.1	1918.1724

Displacement	Base Shear
mm	kN
357.1	1918.1745
357.1	1918.1752
357.1	1918.1759
357.1	1918.1766
357.1	1918.1773
357.1	1918.178
357.1	1918.1787
357.1	1918.1794
357.1	1918.1801
357.1	1918.1808
357.1	1917.8424

FRAME 2.

Table B2: Displacement and base shear values

TABLE: ASCE 41-13 NSP	
Displacement	**Base Shear**
mm	kN
0	0
70	836.7144
133.4	1594.3005
147.4	1735.8129
162.2	1798.7174
167.6	1806.8649
181.9	1817.1428
189.3	1819.8241
199.3	1825.4277
201.5	1823.7467
217.6	1832.1843
220.7	1831.6815
230.9	1836.5937
232.4	1833.48
238.7	1837.0632
240.2	1836.8152
244.4	1837.55
245.3	1836.6579
247.7	1839.3957
291	1858.3231
291	1858.3248
292.5	1858.7607
292.5	1858.7629

Displacement	Base Shear
mm	kN
296.6	1860.1527
296.6	1860.1553
297.2	1860.3699
297.2	1860.3724
297.9	1860.593
297.9	1860.5955
298.2	1860.7055
298.2	1860.708
299	1860.957
299	1860.9596
299.3	1861.0251
299.4	1861.0273
300.6	1861.2323
300.7	1861.235
300.8	1861.2977
300.8	1861.3007
301.3	1861.4945
301.3	1861.4971
301.5	1861.5841
301.5	1861.5866
301.8	1861.6394
301.8	1861.6419
302.8	1861.9843
302.8	1861.9868
303.3	1862.1463
303.3	1862.1486
303.7	1862.2426
303.7	1862.2447
303.7	1862.2447
303.7	1862.247
304.4	1862.4009
305.1	1862.647
305.1	1862.6496
305.4	1862.7614
305.4	1862.764
305.8	1862.8448
305.8	1862.8473
306.5	1863.029
306.5	1863.0316
306.8	1863.154
306.8	1863.1566
307.7	1863.3725
307.7	1863.3747
308.4	1863.5896

Displacement	Base Shear
mm	kN
308.8	1863.711
308.9	1863.7524
308.9	1863.755
309.3	1863.8559
309.3	1863.8585
309.6	1863.9544
309.6	1863.957
309.6	1863.957
309.7	1863.9593
309.7	1863.9594
309.7	1863.9617
310.3	1863.9998

FRAME 3

Table B3: Displacement and base shear values

TABLE: ASCE 41-13 NSP	
Displacement	Base Shear
mm	kN
0	0
70	723.7286
140	1447.2957
180.9	1869.3382
194.4	1998.7134
204.3	2064.4526
207.3	2078.0487
208.7	2078.3321
210.3	2085.4405
216.3	2103.7296
225.3	2126.681
227	2128.8191
228.7	2129.0091
235	2138.6778
237	2139.9615
240.4	2143.7652
245.5	2147.4117
245.5	2147.4194
247.4	2149.1201
247.4	2149.1274
249.6	2150.5077
249.6	2150.5149
252.7	2152.9721

Displacement	Base Shear
mm	kN
257.3	2156.9584
259.2	2155.2572
262.8	2157.396
268	2162.3589
269.9	2162.4402
273.3	2164.282
274.3	2165.0436
279.2	2167.412
281.4	2168.3623
287	2172.5097
287	2172.5162
287.2	2172.6683
287.2	2172.6744
287.2	2172.6756
287.2	2172.6814
287.2	2172.6826
287.2	2172.6885
287.2	2172.6897
287.2	2172.6955
287.2	2172.6967
287.2	2172.7026
287.2	2172.7038
287.2	2172.7096
287.2	2172.7108
287.2	2172.7162
287.2	2172.7177
287.2	2172.7231
287.2	2172.7247
287.2	2172.7301
287.2	2172.7316
287.2	2172.737
287.2	2172.7385
287.3	2172.7439
287.3	2172.7454
287.3	2172.7508
287.3	2172.7523
287.3	2172.7578
287.3	2172.7593
287.3	2172.7647
287.3	2172.7662
288.4	2171.6449
290.2	2173.3137
291.5	2173.3203
295.7	2175.1215

FRAME 4

Table B4: Displacement and base shear values

TABLE: ASCE 41-13 NSP	
Displacement	Base Shear
mm	kN
0	0
70	837.8197
140	1675.4523
152.2	1821.7297
174.5	2049.4878
176.3	2059.2223
179.9	2080.2015
184	2099.1478
189.2	2121.5231
192.9	2131.8274
196.3	2142.342
198.9	2148.5651
208.2	2169.2668
217.6	2181.4584
217.6	2181.4697
219.5	2184.2749
226	2189.7364
226	2189.7435
229.4	2192.5396
229.4	2192.5463
236.9	2199.0708
236.9	2198.532
241.5	2202.5149
241.5	2202.5203
245.3	2205.6217
245.3	2205.6275
251.1	2209.1868
251.2	2209.1913
257.6	2214.5898
257.6	2214.5949
262.1	2217.7057
262.1	2217.7118
264.4	2219.5281

Displacement	**Base Shear**
mm	kN
280.1	2213.9772
327	2245.8726
327.1	2246.6179
327.1	2246.621
334.1	2250.136

APPENDIX C

In this appendix the units are displayed in figure nos. C1-C3

Structure Dimensions				
Absolute Distance	mm			mm
Relative Distance				
Structure Area	mm			mm2
Angles				deg
Section Dimensions				
Length	mm			mm
Area	mm			mm2
Length3	mm			mm3
Length4	mm			mm4
Length6	mm			mm6
Rebar Area	mm			mm2
Rebar Area/Length	mm			mm2/mm
Displacements				
Translational Displ	mm			mm
Rotational Displ				rad
Drift				
Gen Displ L/Rad	mm			mm/rad
Gen Displ Rad/L	mm			rad/mm
Forces				
Force		kN		kN
Force/Length	mm	kN		kN/mm
Force/Area	mm	kN		kN/mm2
Moment	mm	kN		kN-mm
Moment/Length	mm	kN		kN-mm/mm
Temperature			C	C
Temperature Change			C	C
Temperature Gradient	mm		C	C/mm
Stresses				
Modulus	mm	kN		kN/mm2
Stress Input	mm	kN		kN/mm2
Stress Output	mm	kN		kN/mm2
Strain	mm			mm/mm

Figure C1:units

Strain	mm			mm/mm
Stiffness				
Translational Stiffness	mm	kN		kN/mm
Rotational Stiffness	mm	kN		kN-mm/rad
TransRot Coupled Stiff		kN		kN/rad
Trans Stiffness/Length	mm	kN		kN/mm/mm
Rot Stiffness/Length		kN		kN/rad
Trans Stiffness/Area	mm	kN		kN/mm/mm2
Time Related				
Period				sec
Frequency				cyc/sec
Acceleration-Trans	mm			mm/sec2
Acceleration-Rot				rad/sec2
Velocity-Trans	mm			mm/sec
Velocity-Rot				rad/sec
Other Time (Seconds)				sec
Mass and Weight				
Mass	mm	kN		kN-s2/mm
Mass/Length	mm	kN		kN-s2/mm2
Mass/Area	mm	kN		kN-s2/mm3
Mass/Volume	mm	kN		kN-s2/mm4
Weight		kN		kN
Weight/Length	mm	kN		kN/mm
Weight/Area	mm	kN		kN/mm2
Weight/Volume	mm	kN		kN/mm3
Weight*Length2	mm	kN		kN-mm2
Rotational Inertia	mm	kN		kN-mm-s2
Length5	mm			mm5
Modal Factors				
Modal Participation - Trans	mm	kN		kN-mm
Modal Participation - Rot	mm	kN		kN-mm
Modal Stiffness	mm	kN		kN-mm
Participation Mass Ratios				

Figure C2: Units

Modal Mass	mm	kN		kN-mm-s2
Damping Items				
Eff Damping - Trans	mm	kN		kN-s/mm
Eff Damping - Rot	mm	kN		kN-mm-s/rad
Eff Damping - Coupled		kN		kN-s/rad
NL Damping - Trans	mm	kN		kN*(s/mm)^Cexp
NL Damping - Rot	mm	kN		kN-mm*(s/rad)^Cexp
Eff Damping - Trans/Len...	mm	kN		kN-s/mm2
Eff Damping - Trans/Area	mm	kN		kN-s/mm3
Damping Ratio				
Miscellaneous				
1/Length	mm			1/mm
1/Length2	mm			1/mm2
Price/Weight		kN		Price/kN
Energy	mm	kN		kN-mm
Thermal Coefficient			C	1/C
Slider Rate	mm			sec/mm
Demand Capacity Ratio				
Reinforcement Ratio				

Figure C3: Units

REFERENCES

[1] ATC 40 (1996), Seismic Evaluation and Retrofit of Concrete Buildings: Vol. 1 Applied Technology Council, USA.

[2] FEMA 273 (1997). NEHRP Guidelines for the seismic rehabilitation of buildings. *Federal Emergency Management Agency*, Applied Technology Council, Washington D.C., USA.

[3] FEMA 356 (2000), Prestandard and Commentary for the Seismic Rehabilitation of Buildings, American Society of Civil Engineers, USA.

[4] FEMA 440 (2005), Improvement of nonlinear static seismic analysis procedures, Applied Technology Council (ATC) Washington, D.C.

[5] IS 456 (2000). Indian Standard for Plain and Reinforced Concrete - Code of Practice, Bureau of Indian Standards, New Delhi.

[6] IS 1893,"Criteria for Earthquake Resistant Design of structures (part 1) General provisions and buildings (fifth revision)",BIS 2002 Provisions and Buildings

[7] Earthquake Resistant Design of Structures by Pankaj Aggarwal and Manish Shrikhande ,Prentice Hall of India Publication

[8] Earthquake Design Manual for practicing Engineers By Shailesh Kumar Aggarwal C.B.R.I Roorkee

[9] N.Choopool and V. Boonyapinyo Seismic performance evaluation of reinforced concrete moment resisting frames with various ductility in low seismic zone

[10] Prashant Sunagar and S.M Shivananda, Evaluation of seismic response modification factors for R.C.C frames by Non Linear Analysis, Proceedings of International conference on Advances in Architecture and Civil Engineering (AARCV 2012), 21- 23rd June 2012

[11] Iona Olteanu, Ioan-Petru, Ciongradi,Mihaela Anechitei and M.Budescu, The ductile design concept for seismic actions in miscellaneous design codes

[12] S. Talebi And M. R. Kianoush 2, Behaviour Of Reinforced Concrete Frames Designed For Different Levels Of Ductility, 3th World Conference on Earthquake Engineering Vancouver, B.C., and Canada August 1-6, 2004 Paper No. 505

[13] "The Pushover Analysis, explained in its Simplicity", Rahul Leslie1, Assistant Director, Buildings Design, DRIQ Board, Kerala PWD, Trivandrum.

[14] K Rama Raju,A Cinitha, and Nargesh R Iyer Seismic performance evaluation of existing RC buildings designed as per past codes of practice. *Sa¯dhana¯* Vol. 37, Part 2, April 2012, pp. 281–297._c Indian Academy of Sciences

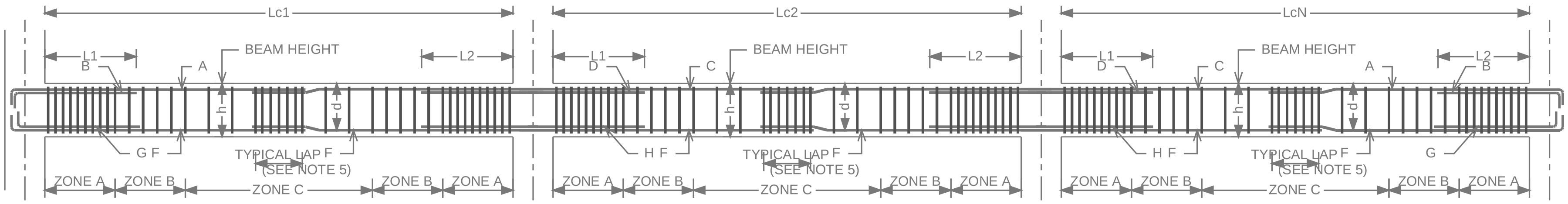
[15] Pushover Analysis of R/C setback frame , A thesis submitted by Rashmita Tripathi, National Institute of Technology Rourkela Orissa -769 008, IndiaAugust 2012.

[16] ASCE/SEI 41-13 American Society of Civil Engineers: seismic evaluation and retrofit of existing buildings**.**

[17] N.K. Manjula, Praveen Nagarajan, T.M. Madhavan Pillai , A Comparison of Basic Pushover Methods. International Refereed Journal of Engineering and Science (IRJES*)*

[18] A. Kiran1, G. Ghosh2 and Y. K.Gupta3 Application of Pushover Analysis Methods for Building Structures Paper No. D002 Indian Society of Earthquake Technology Department of Earthquake Engineering Building IIT Roorkee, October 20-21, 2012

[19] Dr. Graham H. Powell, Professor Emeritus, UC Berkeley Nonlinear dynamic analysis capabilities and limitations.



NOTES:
1. SEE BEAM SCHEDULE FOR STIRRUP TYPE AND SPACING.
2. ZONE A LENGTH SHALL BE TWICE THE BEAM HEIGHT, h.
3. THE FIRST STIRRUP IN ZONE A SHALL BE LOCATED 50.8mm MAXIMUM FROM THE FACE OF THE SUPPORT.
4. LAP SPLICES SHALL NOT BE LOCATED WITHIN THE BEAM/COLUMN JOINT, NOR WITHIN A DISTANCE OF 2H.
5. LAP SPLICES SHALL BE ENCLOSED BY HOOP STIRRUPS AT A MAXIMUM SPACING OF D/4 OR 101.6mm, WHICHEVER IS SMALLER.

Typical Concrete Beam Elevation 3S

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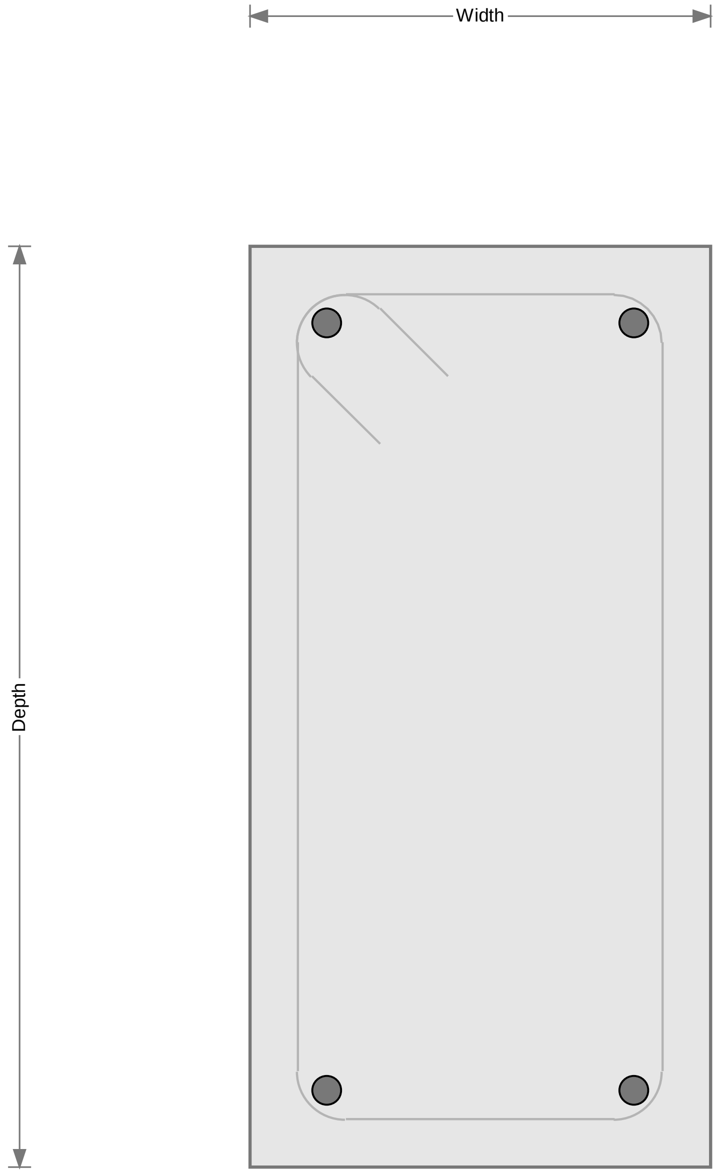

Beam Section-A

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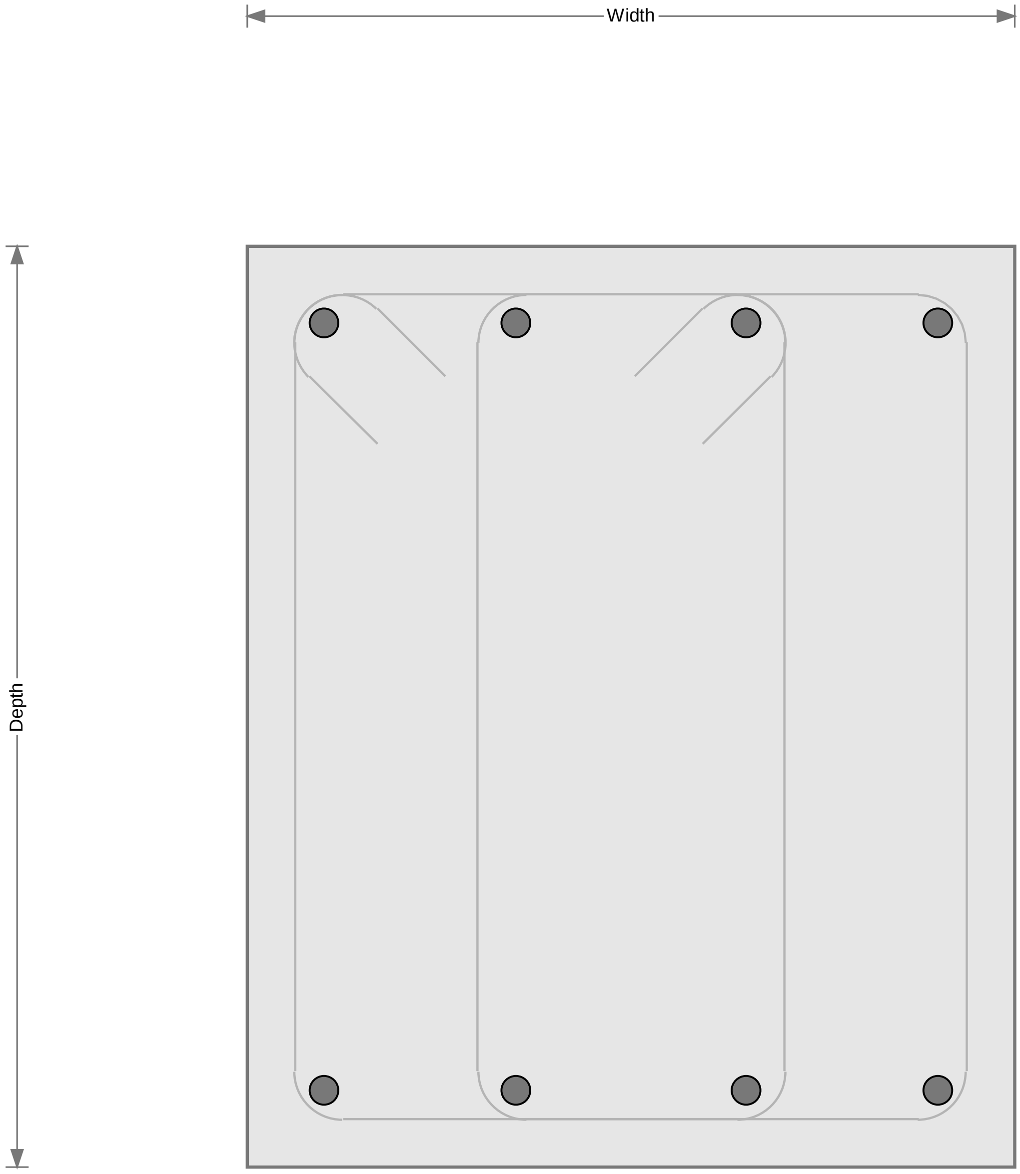

Beam Section-B

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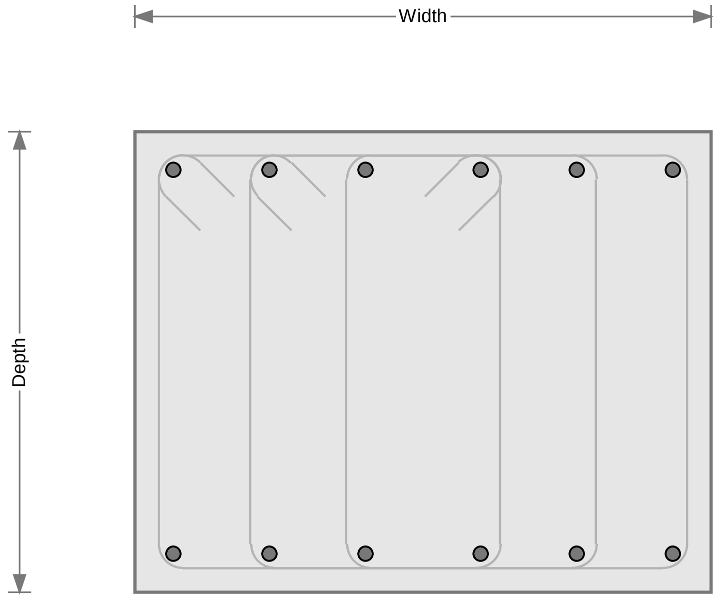

Beam Section-C

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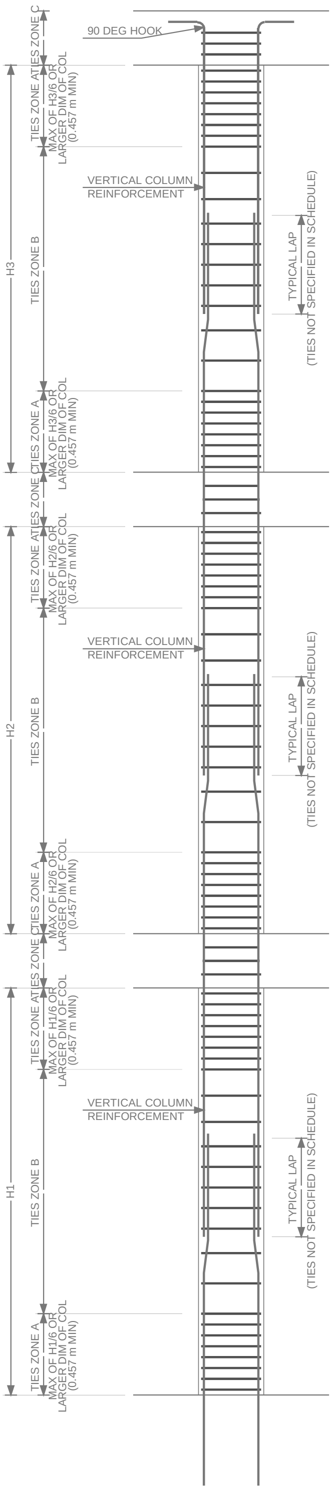

Concrete Column Typical Elevation - Special

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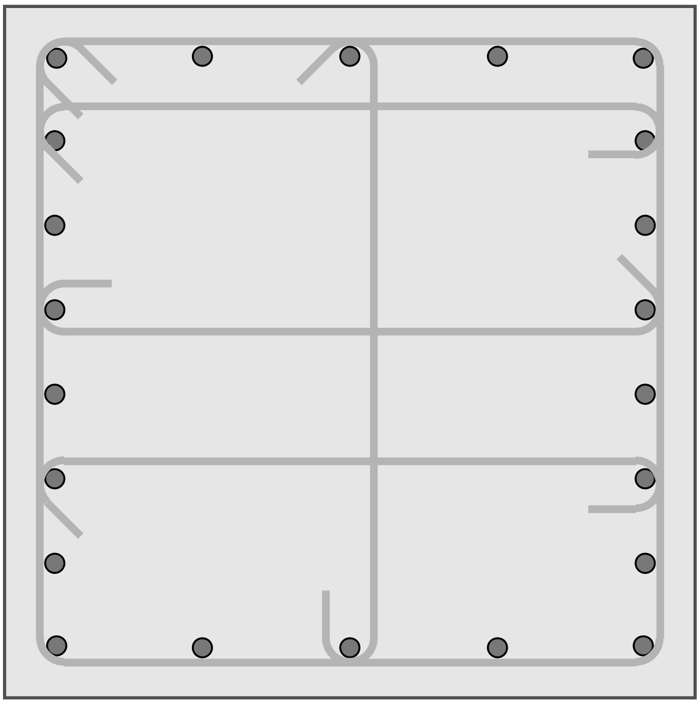

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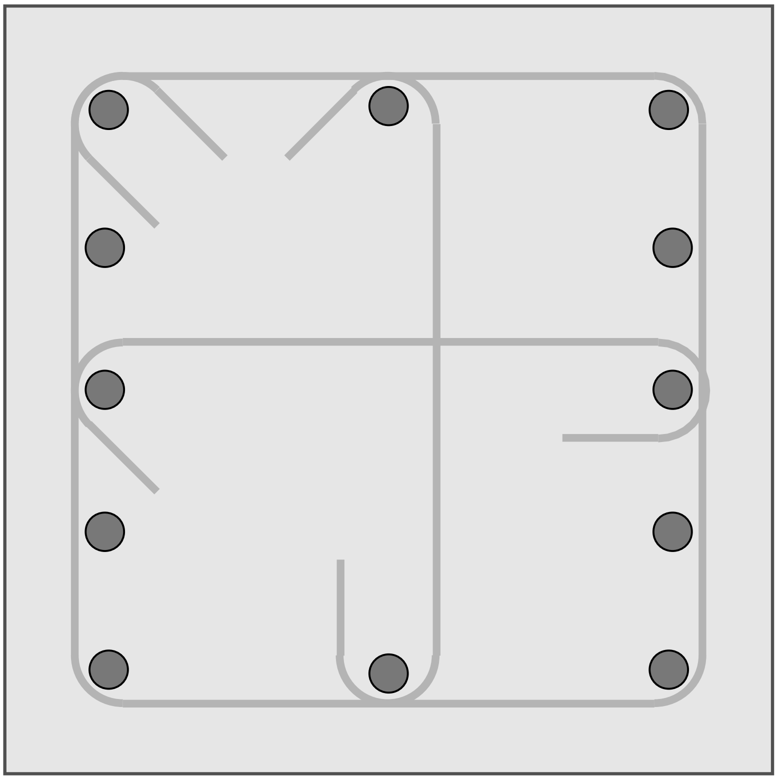

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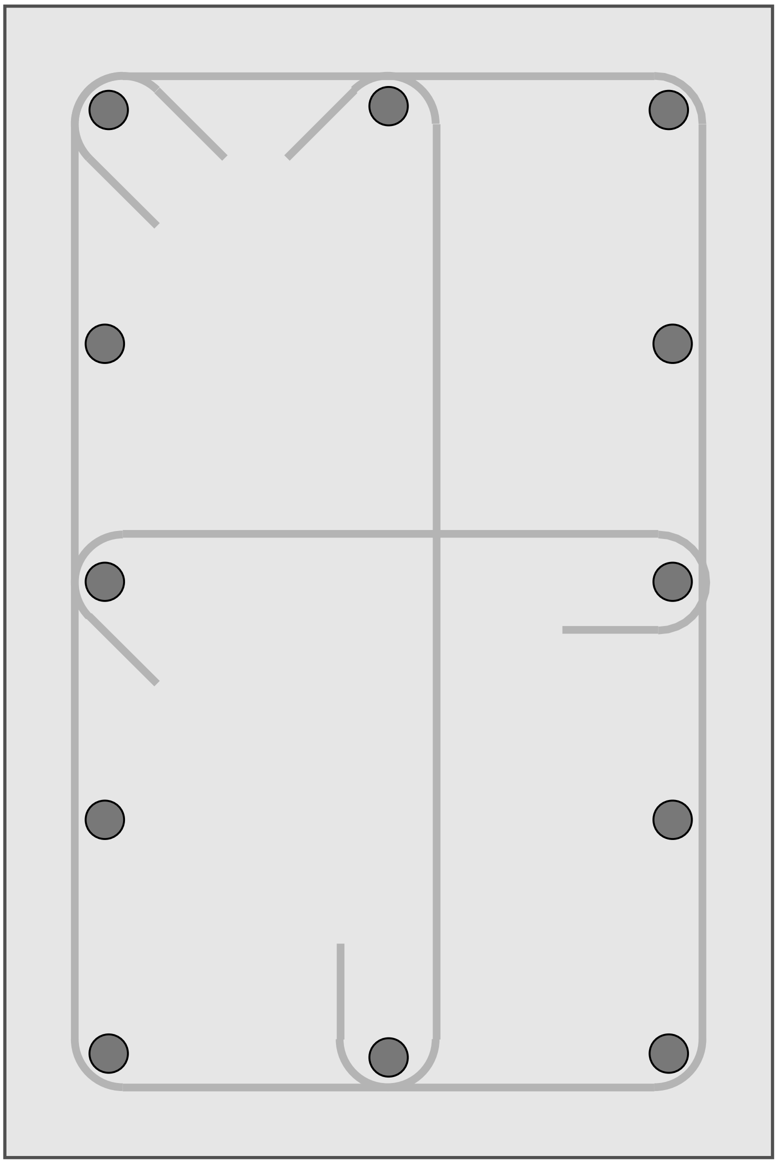

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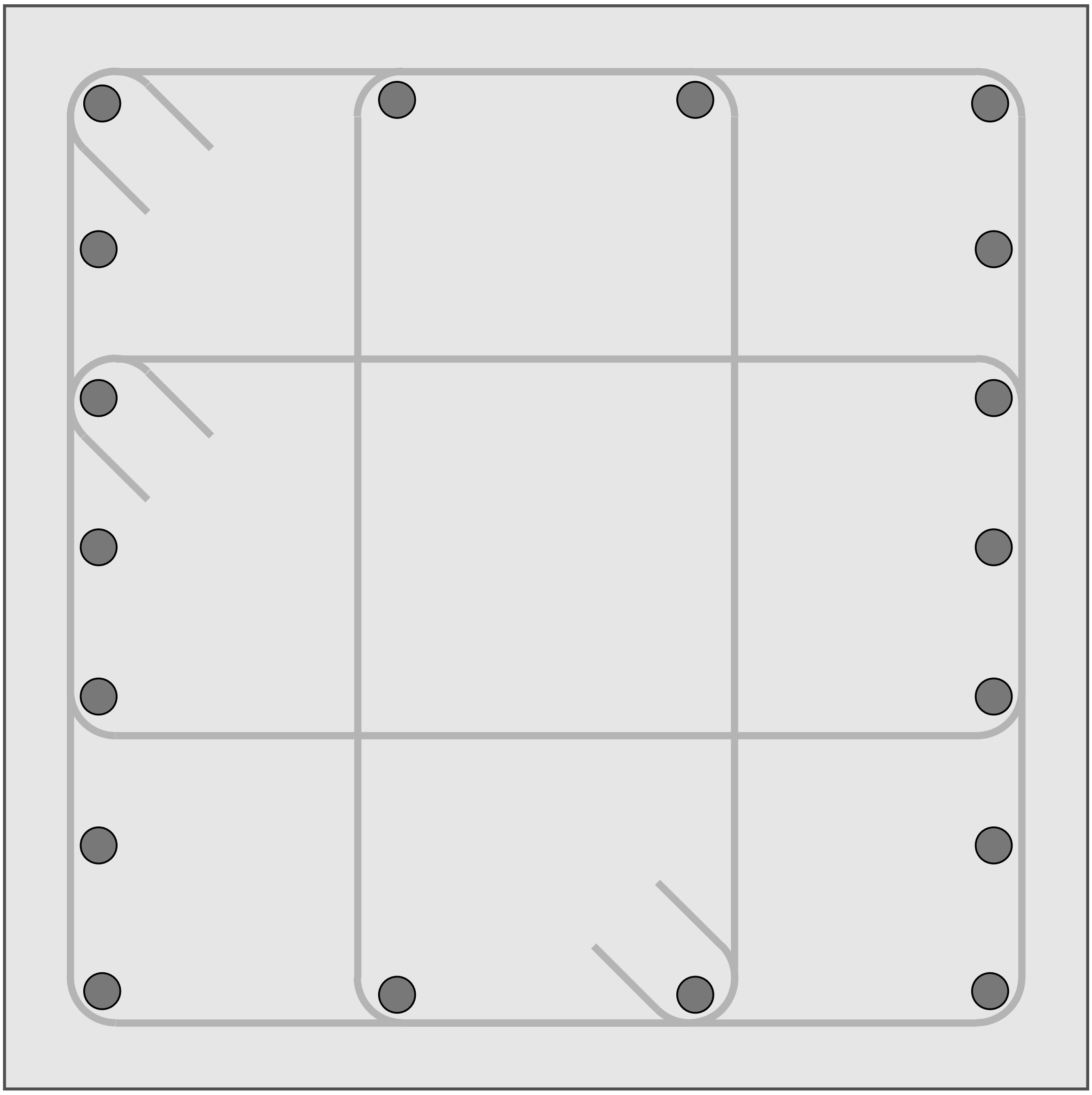

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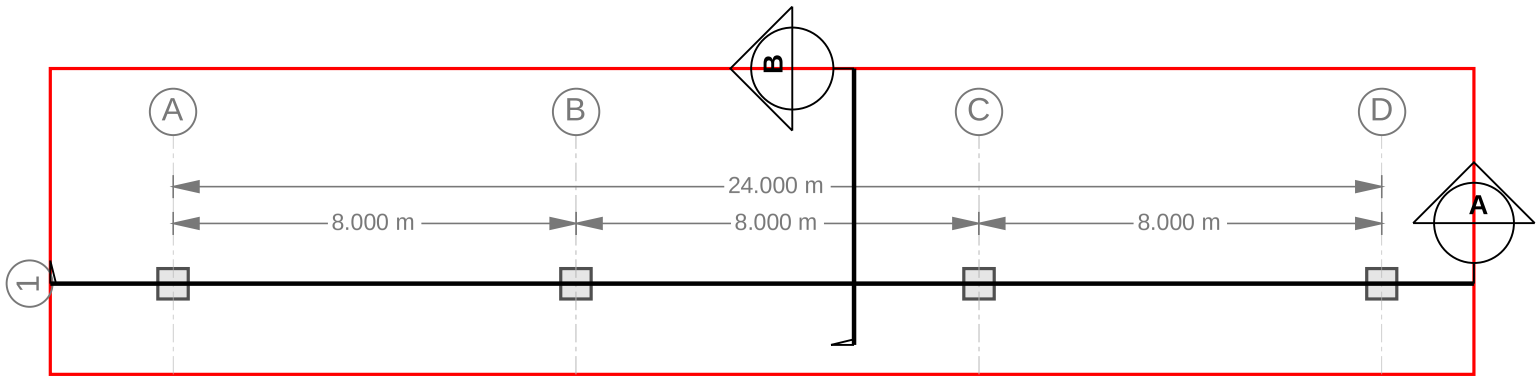

Floor Framing Plan - Base (EL. 0.000 m)
(Scale 1:150)

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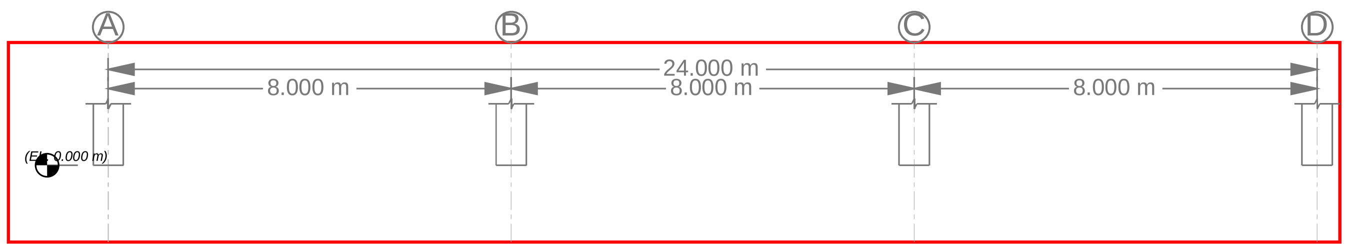

Section A - Base
(Scale 1:150)

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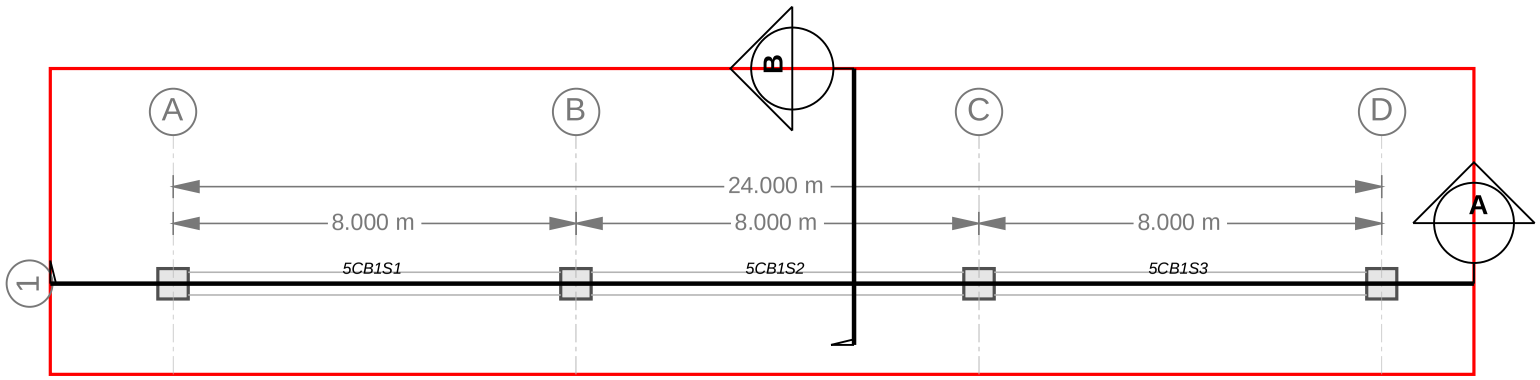

Floor Framing Plan - Story4 (EL. 12.000 m)

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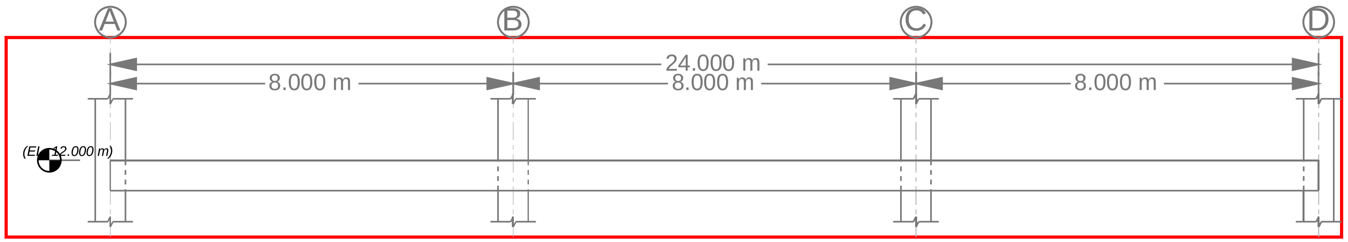

Section A - Story4
(Scale 1:150)

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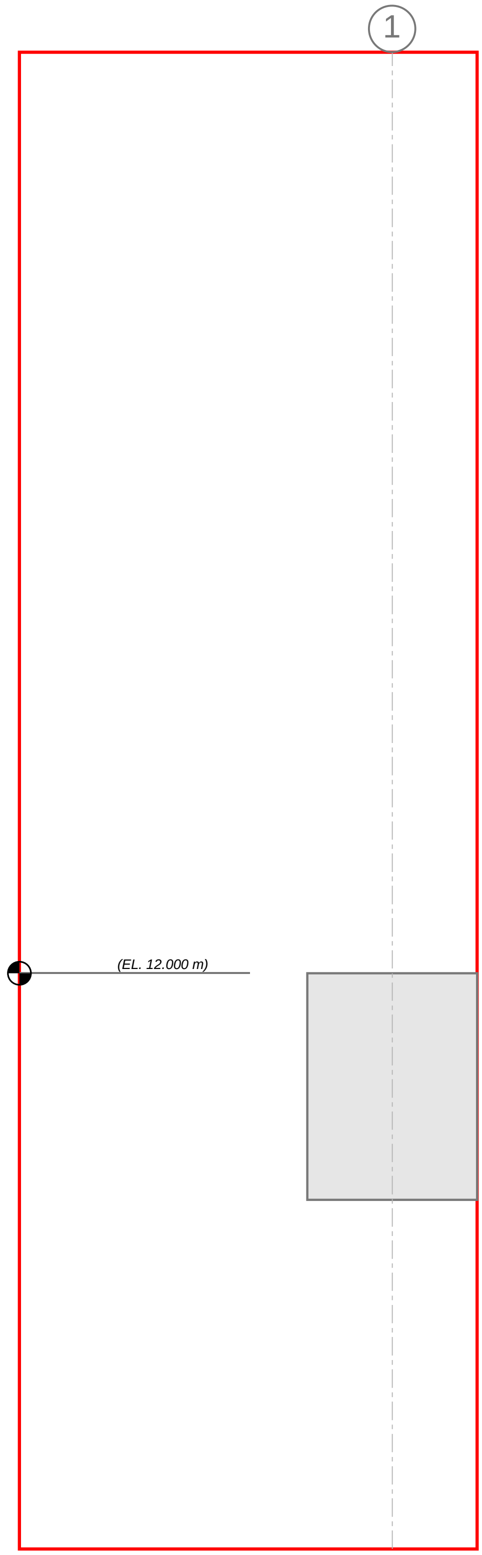
	CONSULTANT	TITLE	DATE	CHECKED	
		SECTION A - STORY4	24-05-2015	APPROVED	IS



Section B - Story4
(Scale 1:20)

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	CONSULTANT	TITLE SECTION B - STORY4	DATE	CHECKED	
			24-05-2015	APPROVED	IS

CONCRETE BEAM REBAR TABLE (SEISMIC)

BEA M ID	SPA N N	SPA N L	SECTIO		LONGITUDINAL BARS							L1	L2	STIRRUPS			TYPICA L ELEV
			WI	DE	A	B	C	D	F	G	H			ZONE A	ZONE B	ZONE C	
12CB 1	1	2.027	450	600	2-20	2-20	-	-	4-20	-	-	0.507	-	4-10 @ 15	-	10 @ 275	ELEVATIO
	2	5.473	450	600	-	-	2-20	2-20	4-20	-	-	-	1.368	4-10 @ 15	-	10 @ 275	ELEVATIO
	3	2.027	450	600	-	-	2-20	2-20	4-20	-	-	-	0.507	4-10 @ 15	-	10 @ 275	ELEVATIO
	4	5.473	450	600	-	-	2-20	2-20	4-20	-	-	-	1.368	4-10 @ 15	-	10 @ 275	ELEVATIO
	5	1.535	450	600	-	-	2-20	2-20	4-20	-	-	-	0.384	4-10 @ 15	-	10 @ 275	ELEVATIO
	6	5.965	450	600	2-20	2-20	2-20	2-20	4-20	-	-	1.491	1.491	4-10 @ 15	-	10 @ 275	ELEVATIO
11CB 1	1	3.012	450	600	2-20	2-20	-	-	4-20	-	-	0.753	-	4-10 @ 15	-	10 @ 275	ELEVATIO
	2	4.488	450	600	-	-	2-20	2-20	4-20	-	-	-	1.122	4-10 @ 15	-	10 @ 275	ELEVATIO
	3	2.519	450	600	-	-	2-20	2-20	4-20	-	-	-	0.630	4-10 @ 15	-	10 @ 275	ELEVATIO
	4	4.981	450	600	-	-	2-20	2-20	4-20	-	-	-	1.245	4-10 @ 15	-	10 @ 275	ELEVATIO
	5	2.519	450	600	-	-	2-20	2-20	4-20	-	-	-	0.630	4-10 @ 15	-	10 @ 275	ELEVATIO
	6	4.981	450	600	2-20	2-20	2-20	2-20	4-20	-	-	1.245	1.245	4-10 @ 15	-	10 @ 275	ELEVATIO
10CB 1	1	3.012	450	600	3-20	3-20	-	-	4-20	-	-	0.753	-	4-10 @ 15	-	10 @ 275	ELEVATIO
	2	4.488	450	600	-	-	2-20	2-20	4-20	-	-	-	1.122	4-10 @ 15	-	10 @ 275	ELEVATIO
	3	3.012	450	600	-	-	3-20	2-20	4-20	-	-	-	0.753	4-10 @ 15	-	10 @ 275	ELEVATIO
	4	4.488	450	600	-	-	2-20	2-20	4-20	-	-	-	1.122	4-10 @ 15	-	10 @ 275	ELEVATIO
	5	2.519	450	600	-	-	3-20	2-20	4-20	-	-	-	0.630	4-10 @ 15	-	10 @ 275	ELEVATIO
	6	4.981	450	600	3-20	3-20	2-20	2-20	4-20	-	-	1.245	1.245	4-10 @ 15	-	10 @ 275	ELEVATIO
9CB1	1	3.504	450	600	4-20	3-20	-	-	4-20	1-20	-	0.876	-	4-10 @ 15	-	10 @ 275	ELEVATIO
	2	3.996	450	600	-	-	2-20	2-20	4-20	-	-	-	0.999	4-10 @ 15	-	10 @ 275	ELEVATIO
	3	3.012	450	600	-	-	4-20	3-20	4-20	-	-	-	0.753	4-10 @ 15	-	10 @ 275	ELEVATIO
	4	4.488	450	600	-	-	2-20	2-20	4-20	-	-	-	1.122	4-10 @ 15	-	10 @ 275	ELEVATIO
	5	3.012	450	600	-	-	4-20	3-20	4-20	-	-	-	0.753	4-10 @ 15	-	10 @ 275	ELEVATIO
	6	4.488	450	600	4-20	3-20	2-20	2-20	4-20	1-20	-	1.122	1.122	4-10 @ 15	-	10 @ 275	ELEVATIO
8CB1	1	3.504	450	600	5-20	4-20	-	-	5-20	1-20	-	0.876	-	4-10 @ 15	-	10 @ 275	ELEVATIO
	2	3.996	450	600	-	-	2-20	2-20	5-20	-	-	-	0.999	4-10 @ 15	-	10 @ 275	ELEVATIO
	3	3.012	450	600	-	-	5-20	3-20	5-20	-	1-20	-	0.753	4-10 @ 15	-	10 @ 275	ELEVATIO
	4	4.488	450	600	-	-	2-20	2-20	4-20	-	-	-	1.122	4-10 @ 15	-	10 @ 275	ELEVATIO
	5	3.012	450	600	-	-	5-20	3-20	5-20	-	1-20	-	0.753	4-10 @ 15	-	10 @ 275	ELEVATIO
	6	4.488	450	600	5-20	4-20	2-20	2-20	5-20	1-20	-	1.122	1.122	4-10 @ 15	-	10 @ 275	ELEVATIO
7CB1	1	3.504	450	600	6-20	5-20	-	-	7-20	1-20	-	0.876	-	4-10 @ 15	-	10 @ 275	ELEVATIO
	2	3.996	450	600	-	-	2-20	2-20	6-20	-	-	-	0.999	4-10 @ 15	-	10 @ 275	ELEVATIO
	3	3.012	450	600	-	-	5-20	5-20	6-20	-	1-20	-	0.753	4-10 @ 15	-	10 @ 275	ELEVATIO
	4	4.488	450	600	-	-	2-20	2-20	5-20	-	-	-	1.122	4-10 @ 15	-	10 @ 275	ELEVATIO
	5	3.012	450	600	-	-	5-20	5-20	6-20	-	1-20	-	0.753	4-10 @ 15	-	10 @ 275	ELEVATIO
	6	4.488	450	600	6-20	5-20	2-20	2-20	6-20	2-20	-	1.122	1.122	4-10 @ 15	-	10 @ 275	ELEVATIO
6CB1	1	7.400	450	600	7-20	6-20	-	-	6-20	4-20	-	1.850	-	4-10 @ 15	-	10 @ 225	ELEVATIO
	2	7.400	450	600	-	-	6-20	6-20	5-20	-	4-20	-	1.850	4-10 @ 15	-	10 @ 250	ELEVATIO
	3	7.400	450	600	7-20	6-20	6-20	6-20	5-20	5-20	4-20	1.850	1.850	4-10 @ 15	-	10 @ 225	ELEVATIO
5CB1	1	7.400	450	600	8-20	6-20	-	-	6-20	5-20	-	1.850	-	4-10 @ 15	-	10 @ 175	ELEVATIO
	2	7.400	450	600	-	-	7-20	6-20	6-20	-	4-20	-	1.850	4-10 @ 15	-	10 @ 175	ELEVATIO
	3	7.400	450	600	8-20	6-20	7-20	6-20	6-20	5-20	4-20	1.850	1.850	4-10 @ 15	-	10 @ 175	ELEVATIO
4CB1	1	7.400	450	600	9-20	7-20	-	-	7-20	5-20	-	1.850	-	4-10 @ 15	-	10 @ 150	ELEVATIO
	2	7.400	450	600	-	-	8-20	7-20	6-20	-	6-20	-	1.850	4-10 @ 15	-	10 @ 175	ELEVATIO
	3	7.400	450	600	9-20	7-20	8-20	7-20	6-20	6-20	6-20	1.850	1.850	4-10 @ 15	-	10 @ 150	ELEVATIO
3CB1	1	7.400	450	600	9-20	7-20	-	-	7-20	6-20	-	1.850	-	4-10 @ 15	-	10 @ 150	ELEVATIO
	2	7.400	450	600	-	-	8-20	7-20	6-20	-	6-20	-	1.850	4-10 @ 15	-	10 @ 175	ELEVATIO
	3	7.400	450	600	9-20	7-20	8-20	7-20	6-20	7-20	6-20	1.850	1.850	4-10 @ 15	-	10 @ 150	ELEVATIO
2CB1	1	3.454	450	600	5-20	4-20	-	-	5-20	1-20	-	0.863	-	4-10 @ 15	-	10 @ 275	ELEVATIO
	2	3.946	450	600	-	-	2-20	2-20	5-20	-	-	-	0.987	4-10 @ 15	-	10 @ 275	ELEVATIO
	3	2.962	450	600	-	-	5-20	4-20	5-20	-	1-20	-	0.740	4-10 @ 15	-	10 @ 275	ELEVATIO
	4	4.438	450	600	-	-	2-20	2-20	4-20	-	-	-	1.110	4-10 @ 15	-	10 @ 275	ELEVATIO
	5	2.962	450	600	-	-	5-20	4-20	5-20	-	1-20	-	0.740	4-10 @ 15	-	10 @ 275	ELEVATIO
	6	4.438	450	600	5-20	4-20	2-20	2-20	5-20	1-20	-	1.110	1.110	4-10 @ 15	-	10 @ 275	ELEVATIO

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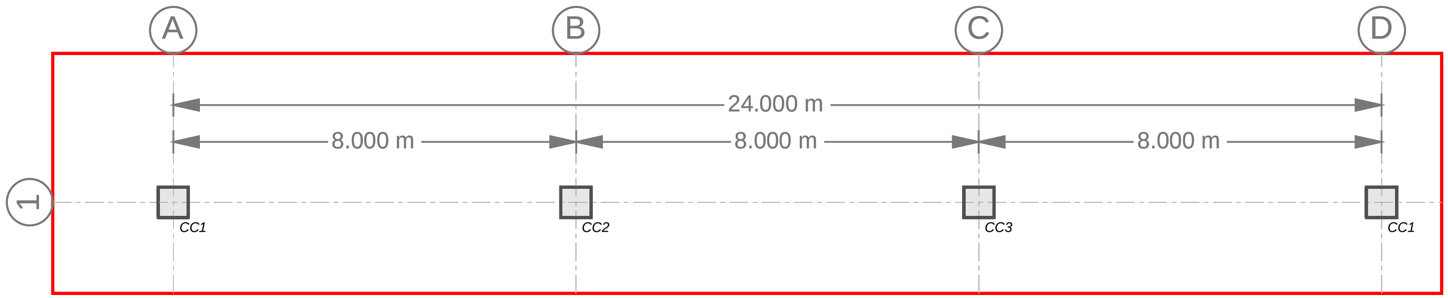
			1	DRAWN	NA
	CONSULTANT	TITLE	DATE	CHECKED	
			24-05-2015	APPROVED	IS



Concrete Column Layout - Base (EL. 0.000 m)
(Scale 1:150)

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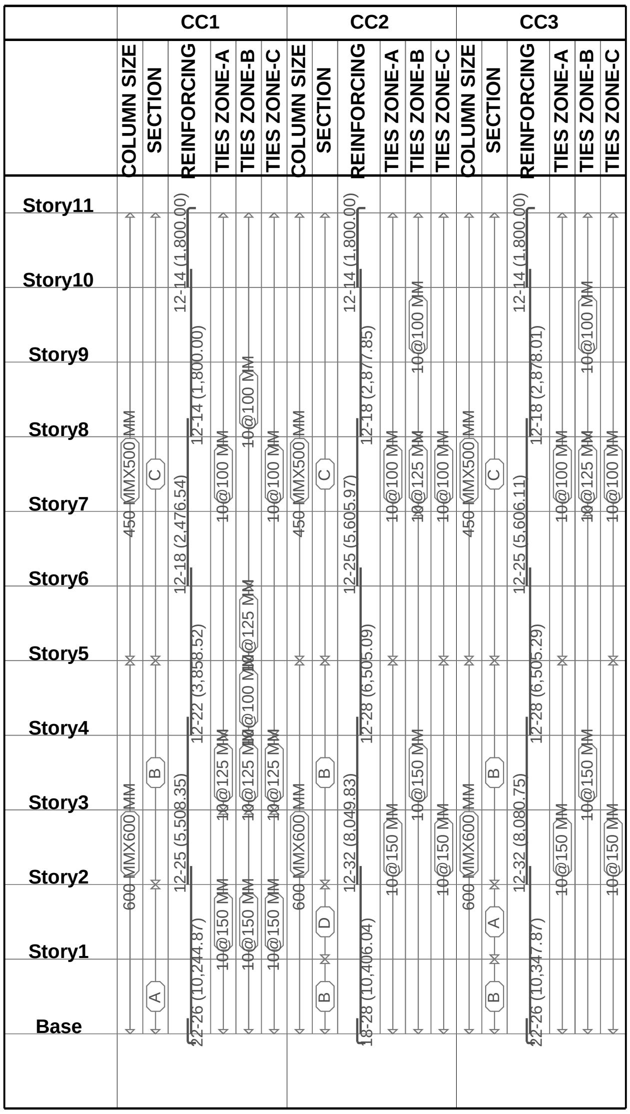
			24-05-2015	APPROVED	IS

Concrete Column Schedule



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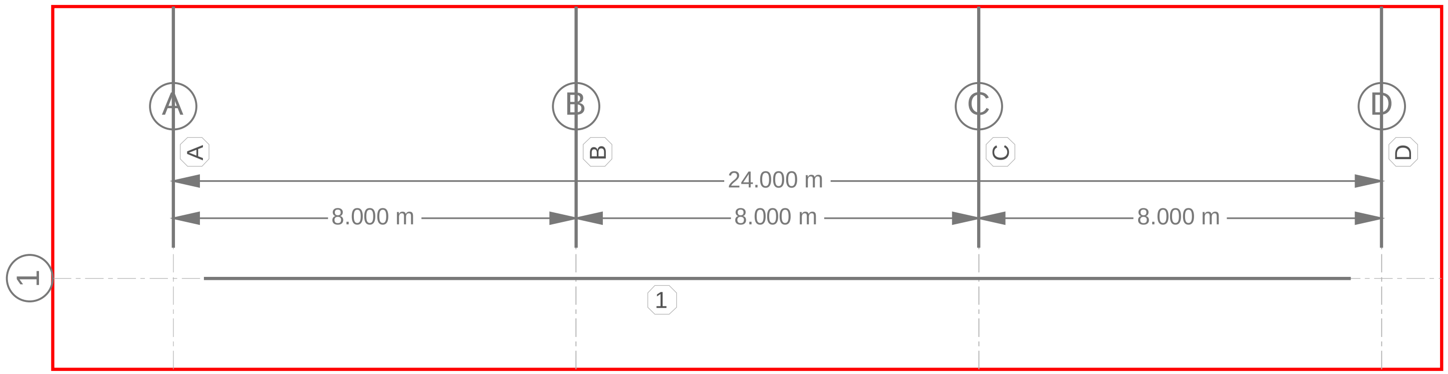

Frame Layout - Base
(Scale 1:150)

	CLIENT	PROJECT	PAGE.	DESIGNED	
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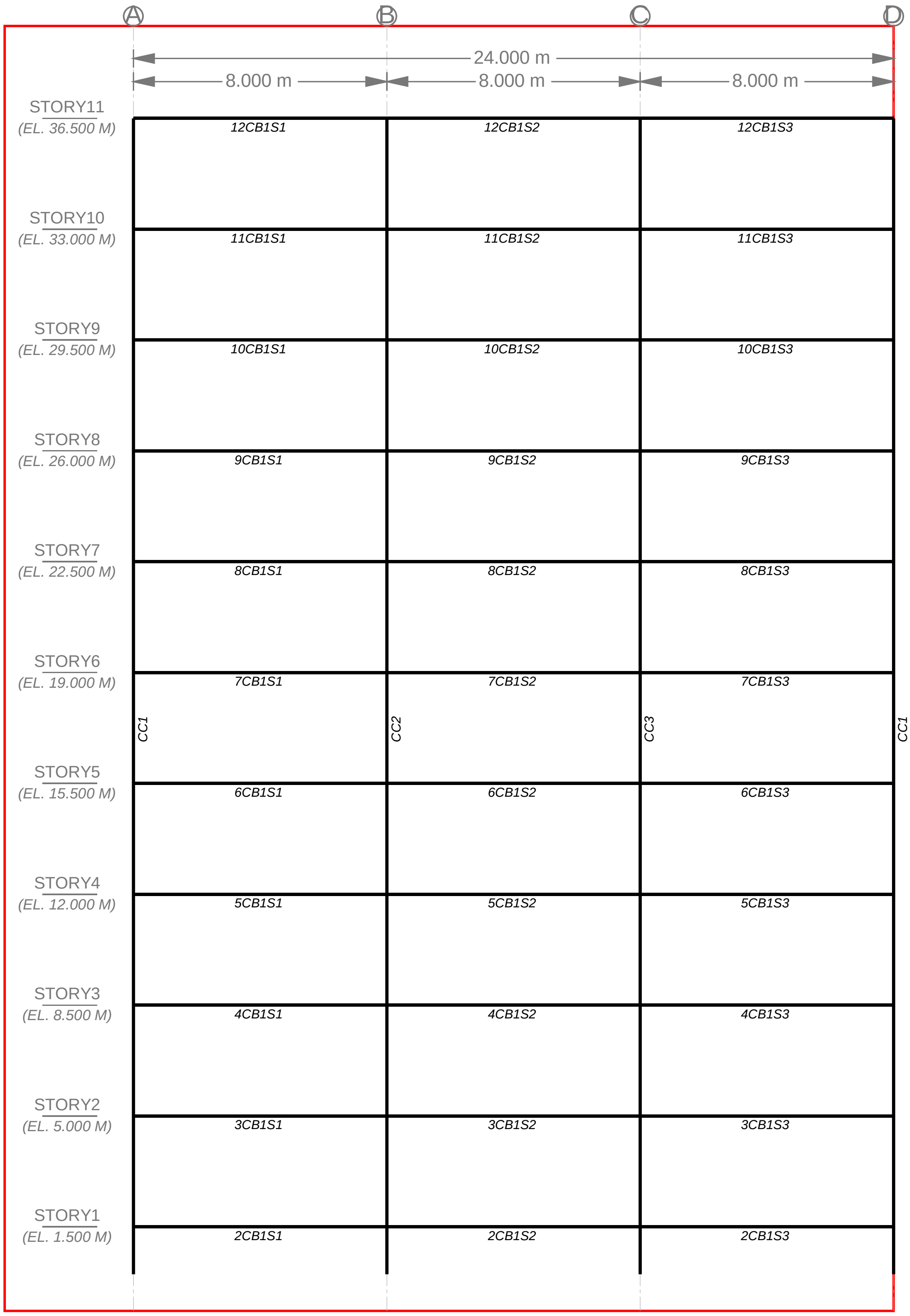
			24-05-2015	APPROVED	IS



1:Elevation

(Scale 1:200)

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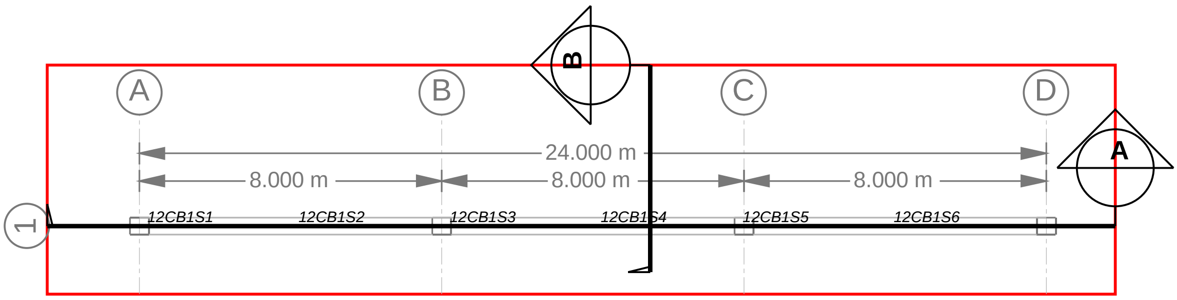

Floor Framing Plan - Story11 (EL. 36.500 m)
(Scale 1:200)

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			24-05-2015	APPROVED	IS