“PUSHOVER ANALYSIS OF 20 STOREY RCC SETBACK BUILDINGS OF DIFFERENT CONFIGURATION USING SAP2000”

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Submitted by

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CANDIDATE’S DECLARATION

I, Chittaranjan Swain, Roll No. 2K13/STE/24 Student of M.Tech. (Structural Engineering), hereby declare that the project Dissertation titled " Pushover Analysis of 20 Storey RCC Setback Buildings of Different Configuration Using SAP2000" which is submitted by me to the Department of Civil Engineering, Delhi Technological University, Delhi in partial fulfilment of the requirement for the award of the degree of Master of Technology, is original and not copied from any source without proper citation. This work has not previously formed the basis for the award of any Degree, Diploma Associateship, Fellowship or other similar title or recognition.

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ABSTRACT

Behaviour of multi-storey framed buildings throughout strong earthquake motion depend on the stiffness, strength and mass distribution in horizontal as well as vertical planes of the buildings. Damage occurring due to earthquake ground motion mainly starts at locations where structural weakness is present in the frames of multi-storey buildings. This weakness further increases and concentrates on the damage of structures by plastification resulting in complete collapse of building. In many cases weakness occurs due to discontinuities in stiffness, mass or strength between two successive storeys. The storey discontinuities are often due to immediate variations in the geometry of frames along with height. In past earthquakes, there are many examples of building failure due to such type of discontinuity in vertical direction. Irregularity in configuration either in elevation or plan was sometimes recognised as one of the main causes building failure during earthquakes. A common type of vertical irregularity (geometrical) in building develops due to sudden reduction in the lateral dimension at specific levels of the building. This type of building is known as setback building. Many investigations has been performed to understand the behaviour of setback buildings and to visualise method for further improvement in performance.

Pushover analysis is a non-linear static analysis mainly used for evaluation of seismic properties of framed buildings conventional pushover analysis outlined in ATC 40:1996 and FEMA 356:2000 is limited for buildings having regular geometry. Using conventional non- linear static pushover analysis, it may not be possible to measure the seismic performance of setback buildings because of the limitation for higher modes effects in irregular structures. There is less research found in the literature for the use of non-linear static (pushover) analysis of setback buildings. The study of conventional method of pushover analysis is instructive.

In the present study a comparative study of non-linear static (pushover) analysis is carried out on different vertically irregular buildings having equal plan area and equal setbacks with different shapes using displacement method.

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CHAPTER-1

INTRODUCTION

1.1 INTRODUCTION

In multi-storey building frames, damages from earthquake ground motion generally starts at locations of structural weakness in the lateral load resisting frames. This behaviour of multi-storey framed buildings during strong earthquakes depend upon the distribution of stiffness, mass and strength in both vertical and horizontal planes of building. In some cases these weakness may be created due to discontinuities in mass, strength or stiffness of subsequent storeys. Such discontinuities between storeys are often associated with variations in the geometry of frame along the height. There are also lots of examples of building failures due to such discontinuities from various previous earthquake data. Structural engineers have developed confidence in the design of buildings having distribution stiffness, strength and mass are more or less uniform. But less confidence is shown in design of structures having irregular geometry.

A common type of vertical irregularity in geometry exists in the presence of setbacks, i.e. due to sudden reduction of the dimension of building laterally at specific levels. These buildings are known as setback building. This type of building form gains increasing popularity in multi-storey building construction now-a-days because of its functional as well as aesthetic architecture. This type of setback firm provides adequate day light and ventilation for lower storeys in an urban locality with a number of tall buildings nearby. This form of building also complies with the norms related to floor area ratio practised in India. Fig 1.1 shows an example of setback building. Change in stiffness and mass along the height render dynamic characteristics differ from regular buildings.

It has been mentioned in literature (Athanassiadou, 2008) that higher mode participation is significant in these buildings. The interstorey drifts in setback buildings are expected to be less in lower floors and more in upper floors as compared to building with regular configurations.

Fig.1.1 A setback building (Location: New Delhi)[*source: www.google.co.in*]

Many investigations have been done to understand the structural behaviour of regular as well as setback buildings and to find method for further improvement of performance. Because of the limitations outlined in FEMA 356(2000) about the conventional non-linear static (pushover) analysis, it may not be possible to evaluate the performance (seismic) of building with setback accurately. In many reports, it is mentioned to extend pushover analysis to include different categories of irregular buildings. However, nothing has been addresses in this regard to setback buildings.

The primary objective of the present study is to study the performance of setback building using conventional pushover analysis method and to suggest necessary improvements in this regard.

1.2 OBJECTIVES OF THE STUDY

For defining the objectives detailed review of literature review is carried out. This is discussed in Chapter -2 in details and summarised briefly here. No design codes have given particular attention to the setback buildings. Research on setback buildings shows that displacement demand depends upon geometrical configuration and concentrated on nearest vicinity of setback in setback buildings. It also mentions significant contribution of higher modes to the response quantities of the structure.

As per description by Presented and Commentary Seismic Rehabilitation of Buildings (FEMA 356:2000); American Society of Civil Engineers, the non- linear static analysis (pushover analysis) is to estimate the seismic demand and capacity of the existing structure. Lateral load is increased monotonically through the building height in this procedure. The building is set to displaced up to the target displacement or until the building collapses. A graphical representation of base shear vs. roof displacement is obtained. This curve is known as capacity curve or pushover curve. The building capacity for an assumed displacement pattern and load distribution is defined by capacity curve. Also, specific state of damage is defined by a point on curve.

Maximum displacement of the building due to earthquake is found by correlating the capacity curve to seismic demand generated by a certain earthquake ground motion. This is called performance point or target displacement. Location of performance point relative to performance levels defines whether performance objective is met or not. As per FEMA 356, it is basically meant for buildings with regular configuration having fundamental modes participation dominant. There are also a number of approaches for pushover analysis mentioned in the literature to make it applicable to regular buildings of different categories. These comprise (i) modal pushover analysis [21] (ii) modified modal pushover analysis [23] (iii) upper bound pushover analysis [32] and (iv) adoptive pushover analysis etc. However, no research has been done on these method's applicability to setback buildings.

Based on the literature review presented, the objective for the present study are mentioned below:

1. To apply pushover method available for their applicability to buildings with setback of different plan and elevation irregularity.

The principle objective of the proposed study is to apply the conventional method (FEMA-356) with conceptual simplicity, but provide more accuracy in seismic demand estimation of setback buildings.

1.3 SCOPE OF STUDY

The present study is limited to multi-storey building frames of reinforced cement concrete with possible setbacks. Setback building models of 20 storeys with irregular plan of equal setback area are taken in consideration. Three buildings having setbacks in all directions are taken.

Plan asymmetry arising due to geometrical irregularity vertically requires three- dimensional analysis for consideration of effects due to torsions. Torsion effect has not been considered in the present study. Storey numbers of 20 storeys with different bay numbers and irregularity are considered. With uniform bay width 4m and height of each storey is restricted to 3m.

For inclusion of effect due to progressive yielding in structure adoptive load pattern should be considered. To keep the procedure simple computational fixed load distribution shapes are planned. Effects of soil structure interactions are not considered in this study.

1.4 METHODOLOGY

Steps considered in the current study to achieve the objectives are as follows:

a) Carry out the review of previous literature extensively, to decide the objectives.
b) Three numbers of building frames with setback are considered. Height of all storeys is taken 3m with widths (4 to 8 bays). Different plans are considered with equal setback above 15th floor level.
c) Analyse the building modes using non- linear static pushover analysis.
d) Perform a comparative study on the setback building frames.

1.5 ORGANISATION OF THE THESIS

Chapter 1 presents the background, objective, scope and methodology followed.

Chapter 2 represents the previous work carried out on the moment resisting setback frames by various researchers. A detailed description of pushover analysis as per FEMA 356 and ATC 40 are also presented with references to its limitations.

Chapter 3 includes analytical modelling which is done for the representation of actual behaviour of structural components of building frames. In this chapter plans for setback of buildings of different geometry (square, rectangular, and L-type) are also explained.

Chapter 4 starts with the presentation of general behaviour of building due to earthquake ground motion. Modelling of plastic hinges is also discussed in details. Pushover curves are also drawn for setback buildings. Finally a comparison has been carried out between the buildings with setback of different geometry in accordance with the non- linear static (pushover) analysis.

Chapter 5 includes significant discussions and conclusions drawn from the study carried out and the further scope for the research.

CHAPTER-2

LITERATURE REVIEW

2.1 INTRODUCTION

Literature review is carried out on the performance of setback building under seismic loading. First half describes the literature published on the setback building frames. It also describes a number of analytical and experimental works on setback buildings. The second half devoted to a detailed study on pushover analysis methods. Non- linear static analysis methods published in ATC 40(1996) [3] report together with FEMA 356(2000) [16] report are explained. Procedure for pushover analysis as per FEMA 356 and ATC 40 is presented. The change in modal properties due to progressive yielding of building component is not considered. In recent literature review, there have been a number of attempts published to extend the pushover analysis to take higher mode effects in account [11][12][23][26]. Consideration of progressive structural yielding using adoptive procedures has also included with updated force distribution which has been taken into account in the current state of stiffness and strength of the frames of building at each step [5][17][28]. In the end of this chapter the major drawbacks of current pushover analysis procedures has been discussed and also to overcome the drawbacks some selected alternative pushover analysis procedure are studied from the literature.

2.2 RESEARCH ON SETBACK BUILDINGS

Experimental and analytical investigations have been carried out by a number of researchers to identify the main differences in the dynamic response of regular and setback buildings. Mainly, the study on the displacement response and ductility demands has been focussed.

A study has been carried out on the inelastic seismic response of plane sheet moment resisting frames with setbacks [19]. In the research, in order to drive the structures to different limit states, a group of 120 frames, has been designed in accordance to the European Seismic and Structure codes, is subjected to ensemble of 30 ordinary earthquake ground motions at different intensities. The author came to a conclusion that the geometrical configuration and the level of inelastic deformation play a major role on the height wise distribution of deformation demands. Also, in the neighbourhood of setbacks for other geometrical configuration, the maximum deformation demands are concentrated in the “tower” for tower type structures.

Another study addressed the effects of setbacks on the earthquake response of multi-storey buildings [30]. To improve the design methods of setback structures, an effort was undertaken which include an experimental and analytical study. The experimental study includes a 6-storey moment resisting reinforced concrete space structure with 50% setback in one direction at mid height. Analytical study was primarily focussed in the test structures. Over the height the displacement profiles were relatively smooth. At the tower-based junction relatively large inter-storey drifts were followed by a moderate increase in damage at that level. Overall, from the displacement and inertia force profiles, the predominance of the fundamental mode on the global translational response in the direction parallel to the setbacks was clear. Almost the distribution of lateral forces was similar to the distribution specified by the UBC codes; in dynamic process no significant peculiarities were detected. For further investigation, an analytical study was done on 6- generic reinforced concrete setback frame buildings.

Seismic performance of multi-storey reinforced concrete (R.C.C) building frames with irregularity in elevation has been proposed in a paper [4]. The author has designed two 10-storey 2-D plane frames along with two and four large setbacks in the upper floors respectively, as well as a 3^{rd} one, which is considered to be regular in elevation, in provision of the 2004 Eurocode 8(EC8)[15]. For selected input motions, all frames are subjected to both inelastic static pushover analysis and inelastic dynamic time-history analysis. The effect of ductility class on the cost of building is negligible is the conclusion drawn from the above. Also, in the upper floors of the irregular frames, conventional pushover analysis seems to be underestimating the response quantities. Seismic performance of setback frames are not inferior (and satisfactory) to that of the regular ones even for the motion twice as strong as the design earthquakes. From the above mentioned reference, the setback buildings and regular buildings designed in accordance to EC-8 perform equally well when subjected to seismic loadings.

The studies on the seismic behaviour of the vertically irregular structures along with their findings in the building codes has been reviewed along with available literature and the knowledge of the seismic response of vertically irregular building frames has been summarised [33]. To classify the vertical irregular structures a criteria has been provided by using the building codes and dynamic analysis suggest to arrive at design lateral force. Author observed that most of the studies emphases on the increase in drift demand in the tower portion of setback structure and increase in seismic demand for building frames with discontinuous distribution in strength, mass and stiffness. For the combined strength and stiffness the largest seismic demand is found.

The validity of design code requirement for setback buildings which requires a dynamic analysis with the base shear calibrated by the static base shear obtained using the code's equivalent static load procedure [31]. Mainly two major issues has been discussed in the paper which includes (i) whether the code static base shear is applicable to setback buildings and (ii) whether for computing the base shear the higher mode period should be used when the modal-weight of a higher mode is larger than that of fundamental mode. For adjusting the code period formula, modification factors were derived so that it can provide a good reasonable estimate for the period of a building with setbacks. Using the higher mode period for base shear calculations, different cases were demonstrated for whether the modal weight of a higher mode is larger than that of fundamental mode, which will result in unnecessarily conservative design.

2.3 PUSHOVER ANALYSIS : AN OVERVIEW

The use of non-linear static (pushover) analysis came in to practice in 1970's but the potential has been recognised for last 10-15 years. To estimate the drift capacity and strength of existing structure and the seismic demand for the structure subjected to selected earthquake, the procedure is mainly used. The pushover analysis procedure can also be used for checking the sufficiency of new structural design.

An analysis where a building frame model directly incorporating the non-linear load deformation characteristic of particular components shall be subjected to monotonically increasing lateral loads representing inertial forces in earthquakes until a 'target displacement' is exceeded, is known as pushover analysis. Where the target displacement is defined as the maximum displacement of the building frame at roof expected under selected earthquake ground motion. Using the non-linear static (pushover) analysis algorithm the structural performance can be assessed by estimating the force and deformation capacity and seismic demand. Storey drifts, storey forces, component deformation, component forces, and global displacements (at roof/other reference point) are the seismic demand parameters used. The analysis also explains the redistribution of internal forces and geometrical non linearity material inelasticity. From the pushover analysis, following response characteristics can be obtained which is summarised below:

a) Estimation of displacement capacities and force of the structure. Sequencing the progress of the overall capacity curve and member yielding.
b) Under the earthquake ground motion, estimation of global displacement demand, corresponding inter-storey drift and damages on structural and non-structural elements.
c) Estimation of force (shear, moment and axial) demand on potentially brittle elements and deformation demands on ductile elements.
d) On the overall structural stability, sequencing of the failure of elements and the consequent effect.
e) Identification of strength irregularities (in plan/in elevation) of the building and identification of the critical region, where the inelastic deformation are expected to be high.

Over the linear static analysis, non-linear static (pushover) analysis presents all these above-mentioned benefits for an additional computational effort (modelling non-linearity and change in analysis algorithm).

2.3.1 Pushover Analysis Procedure

Pushover analysis is a non-linear static procedure along the height of building in which the magnitude of the lateral load is increases monotonically maintaining a predefined distribution pattern (Fig 2.1(a)). Throughout the procedure, the sequence of cracking ,plastic hinging and failure of the structural components is observed. Building is displaced till the 'control node' reaches 'target displacement' or building collapses. For all pushover analysis, a curve representing the relation between base shear and control node displacement is plotted (Fig 2.1(b)). Conventionally this particular curve is called pushover curve or capacity curve which is the most important part of non-linear static(pushover) analysis.

Estimation of the 'target displacement' cab be done by the capacity curve. Therefore, the pushover analysis may be carried out twice: (a) to estimate the target displacement till the collapse of the building and (b) to estimate the seismic demand till the target displacement. For the selected earthquake, the seismic demands (storey drifts, storey forces and component deformation and forces) are calculated at the target displacement level. To know what performance the structure will exhibit, the seismic demand is then compared with the corresponding structural capacity or predefined performance limit state.

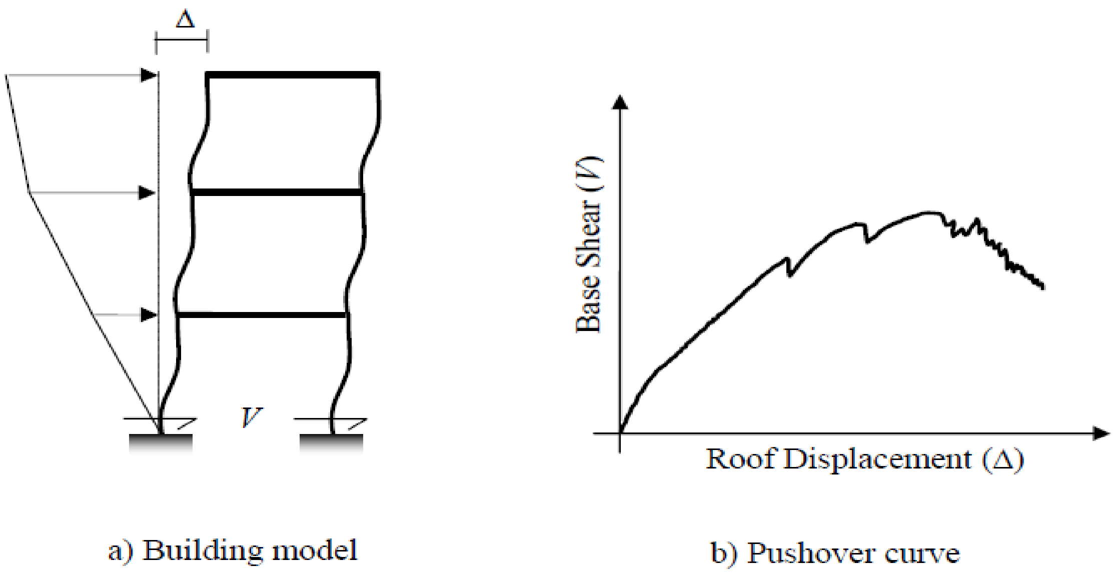

Fig 2.1 Schematic representation of pushover analysis procedure [18]

The analysis results are sensitive to the selection of the lateral load pattern and control node. In general, the control node is the centre of mass location at the roof of the building. In pushover analysis, for selecting lateral load pattern a set of guidelines as per FEMA 356 is explained. To study the actual behaviour, in both positive and negative direction the lateral load is applied in combination with gravity load (dead load and live load).

2.3.2 Lateral Load Profile

In pushover analysis of setback building, along the height of the building, the building is pushed with a specific load distribution pattern. Till the end of the process, the magnitude of the total force is increased but the pattern of the loading remains same. During an earthquake, the lateral load pattern should be approximate the inertial forces as expected in the building. Within the structure, the distribution of lateral inertial forces determines the reactive magnitude of moments, shears and deformation. During earthquake response, the distributions of these inertial forces will continuously vary as the members yield and stiffness characteristics changes. Also, it depends on the type and magnitude of earthquake ground motion. FEMA 356 recommends primarily invariant load pattern. For pushover analysis of R.C.C frames building, although the inertial force distribution vary with the severity of earthquake and with time.

A triangular or trapezoidal shape of lateral load provide a better fit to dynamic analysis results at the elastic range have been found in several investigations [17][24], but at large deformation the dynamic envelopes are closer to the uniformly distributed force pattern. For all pushover analysis, FEMA 356 suggests the use of at least two different patterns.

FEMA 356 recommends selecting load pattern from each of the mentioned two groups below:

(a) Group-I:

- In equivalent static analysis code based vertical distribution of lateral forces are used(permitted only when more than 75% of the total mass take part in the fundamental mode in the direction under consideration)
- A vertical distribution proportional ti the story shear distribution calculates by the combination of modal response from a response spectrum analysis of the building (consideration of sufficient number of modes to capture 90% of the total building mass). When the period of the fundamental mode exceeds 1.0 second this distribution shall be used.
- A vertical distribution proportional to the shape of fundamental mode in the direction under consideration.

(b) Group-II:

- A uniform distribution consisting of lateral forces at each level proportional to the total mass at each level.
- An adaptive load distribution that changes as the structure is displaced. Modification of the adaptive load distribution shall be done from the original load distribution by using a method that considers the properties of the yielded structure.

To bind the solution, instead of using the uniform distribution, FEMA 356 also allows adaptive lateral load patterns are used. Adaptive procedure may yield results that are more accurate with the characteristics of the building under consideration, but it requires considerably more analysis effort.

The common lateral load pattern used in pushover analysis has been shown in Fig 2.2.

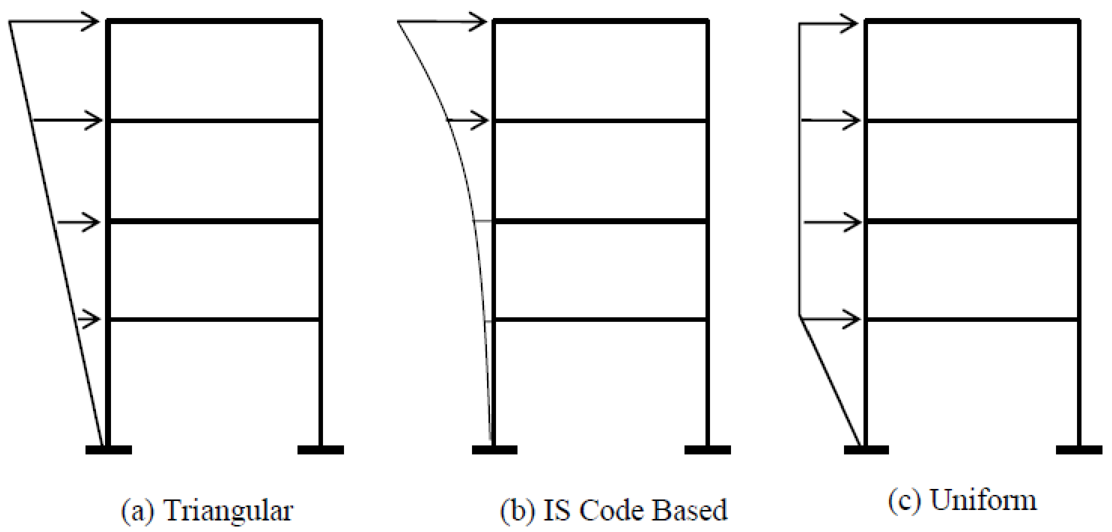

Fig 2.2 Lateral load pattern for pushover analysis as per FEMA 356 (considering uniform mass distribution) [16]

2.3.3 Target Displacement

The displacement demand for the building at control node subjected to the ground motion under consideration is known as target displacement. To know the building performance, in pushover analysis the target displacement plays a important parameter because the global and component response (displacements and forces) of the building at the target displacement are compared with the desired performance limit state. So, on the accuracy of target displacement the success of pushover analysis is largely dependent. To calculate target displacement, there are mainly two approaches:

- Displacement coefficient method (DCM) of FEMA 356
- Capacity spectrum method (CSM) of ATC 40

For the calculation of global displacement demand on the building both the approaches use pushover analysis curve,from the response of an equivalent shingle-degree-of-freedom (SDOF) system.

Displacement Coefficient Method

Primarily this method estimates the elastic displacement of an equivalent SDOF system assuming initial linear properties and damping for the ground motion excitation under consideration. Then the estimation of the total maximum inelastic displacement response for the building at roof by multiplying with a set of displacement coefficients.

The method begins with the pushover curve (base shear vs. roof displacement) as shown in Fig 2.3(a). By graphical procedure an equivalent period (T_{eq}) is generated from the initial period (T_i). This equivalent period represents the linear stiffness of the equivalent SDOF system. Calculation of the peak elastic spectral displacement corresponding to this period is can be directly done from the response spectrum representing the seismic ground motion under consideration (Fig 2.3(b)).

$$S_d = \frac{T_{eq}^2}{4\pi^2} S_a$$

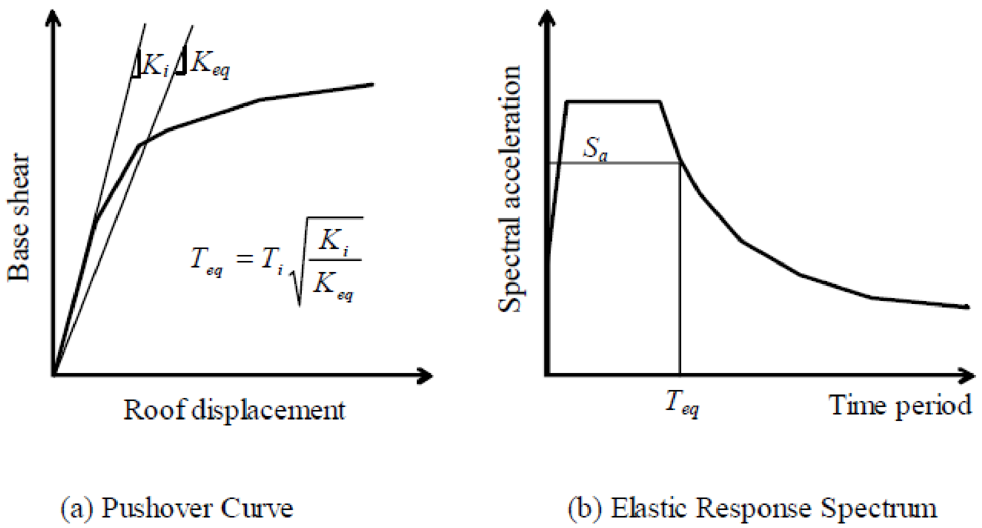

Fig. 2.3 Schematic presentation of displacement coefficient method [16]

Under the selected seismic ground motion, the expected maximum roof displacement of the building (target displacement) can be expressed as:

$$\delta_t = C_0 C_1 C_2 C_3 S_d = C_0 C_1 C_2 C_3 \frac{T_{eq}^2}{4\pi^2} S_a \qquad (2.2)$$

Where, C_O = a shape factor to convert the spectral displacement of equivalent SDOF system to the displacement at the roof of building frame.

C_1 = the ratio of expected displacement for an inelastic system to displacement of linear system.

C_2 = a factor that accounts the effect of pinching in load deformation relationship due to strength and stiffness degradation.

C_3 = a factor to adjust geometric nonlinearity (P-Δ) effects.

S_a = spatial acceleration

From the above definition of the coefficients, the change in geometry of building will affect C_O significantly whereas it is likely to have very little influence on the other factors. From FEMA 356, the value of C_O factor for shear buildings depends only on the number of storeys and the lateral load pattern used in the pushover analysis. The values of C_O provided FEMA 356 for shear building frames has been presented in Table 2.1. From the table mentioned below, setback buildings have 5 or more storey have constant C_O factor according to FEMA 356.

Table 2.1 values of C_O factor for shear buildings

Number of storeys	Triangular load pattern	Uniform load pattern
1	1	1.00
2	1.2	1.15
3	1.2	1.20
5	1.3	1.20
10+	1.3	1.20

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 (Fig. 2.4).

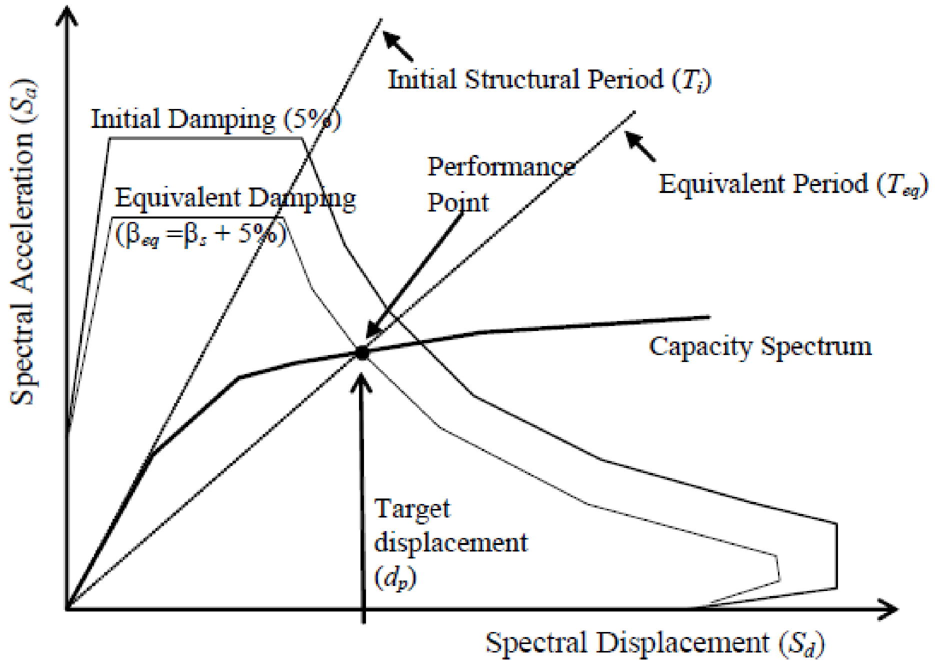

Fig. 2.4: Schematic representation of Capacity Spectrum Method (ATC 40)

The equivalent period (T_{eq}) is computed from the initial period of vibration (T_i) of the nonlinear system and displacement ductility ratio (μ). Similarly, the equivalent damping ratio (β_{eq}) 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 (T_{eq}) and equivalent damping (β_{eq}).

$$T_{eq} = T_i \sqrt{\frac{\mu}{1 + \alpha\mu - \alpha}} \tag{2.3}$$

$$\beta_{eq} = \beta_i + \kappa \frac{2}{\pi} \frac{(\mu - 1)(1 - \alpha)}{\mu(1 + \alpha\mu - \alpha)} = 0.05 + \kappa \frac{2}{\pi} \frac{(\mu - 1)(1 - \alpha)}{\mu(1 + \alpha\mu - \alpha)} \tag{2.4}$$

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.

ATC 40 relates effective damping to the hysteresis curve (Fig. 2.5) and proposes three hysteretic behaviour types that alter the equivalent damping level. Type A hysteretic behaviour is meant for new structures with reasonably full hysteretic loops, and the corresponding equivalent damping ratios take the maximum values. Type C hysteretic behaviour represents severely degraded hysteretic loops, resulting in the smallest equivalent damping ratios.

Type B hysteretic behaviour is an intermediate hysteretic behaviour between types A and C. The value of κ decreases for degrading systems (hysteretic behaviour types B and C).

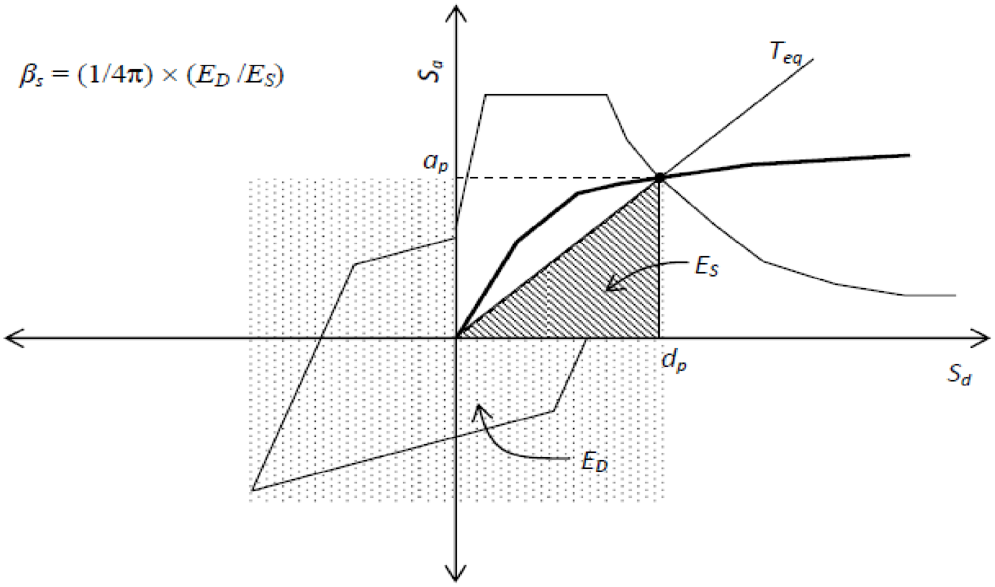

Fig. 2.5: Effective damping in Capacity Spectrum Method (ATC 40)

The equivalent period in Eq. 2.3 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.

ATC 40 provides reduction factors to reduce spectral ordinates in the constant acceleration region and constant velocity region as a function of the effective damping ratio. The spectral reduction factors are given by:

$$SR_A = \frac{3.21 - 0.68\ln(100\beta_{eq})}{2.12} \tag{2.5}$$

$$SR_V = \frac{2.31 - 0.41\ln(100\beta_{eq})}{1.65} \tag{2.6}$$

Where β*eq* is the equivalent damping ratio, *SRA* is the spectral reduction factor to be applied to the constant acceleration region, and *SRV* is the spectral reduction factor to be applied to the constant velocity region (descending branch) in the linear elastic spectrum.

Since the equivalent period and equivalent damping are both functions of the displacement ductility ratio (Eq. 2.3 and Eq. 2.4), 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.

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2.4 SHORTCOMINGS OF THE PUSHOVER ANALYSIS

Pushover analysis is a very effective alternative to nonlinear dynamic analysis, but it is an approximate method. Major approximations lie in the choice of the lateral load pattern and in the calculation of target displacement. FEMA 356 guideline for load pattern does not cover all possible cases. It is applicable only to those cases where the fundamental mode participation is predominant. Both the methods to calculate target displacement (given in FEMA 356 and ATC 40) do not consider the higher mode participation. Also, it has been assumed that the response of a MDOF system is directly proportional to that of a SDOF system. This approximation is likely to yield adequate predictions of the element deformation demands for low to medium-rise buildings, where the behaviour is dominated by a single mode. However, pushover analysis can be grossly inaccurate for buildings with irregularity, where the contributions from higher modes are significant. Many publications [16,19,23,25]. Significant shortcomings and limitations, which are summarized below:

a) One important assumption behind pushover analysis is that the response of a MDOF structure is directly related to an equivalent SDOF system. Although in several cases the response is dominated by the fundamental mode, this cannot be generalised. Moreover, the shape of the fundamental mode itself may vary significantly in nonlinear structures depending on the level of inelasticity and the location of damages.
b) Target displacement estimated from pushover analysis may be inaccurate for structures where higher mode effects are significant. The method, as prescribed in FEMA 356, ignores the contribution of the higher modes to the total response.
c) It is difficult to model three-dimensional and torsional effects. Pushover analysis is very well established and has been extensively used with 2-D models. However, little work has been carried out for problems that apply specifically to asymmetric 3-D systems, with stiffness or mass irregularities. It is not clear how to derive the load distributions and how to calculate the target displacement for the different frames of an asymmetric building. Moreover, there is no consensus regarding the application of the lateral force in one or both horizontal directions for such buildings.

d) The progressive stiffness degradation that occurs during the cyclic nonlinear earthquake loading of the structure is not considered in the present procedure. This degradation leads to changes in the periods and the modal characteristics of the structure that affect the loading attracted during earthquake ground motion.

e) Only horizontal earthquake load is considered in the current procedure. The vertical component of the earthquake loading is ignored; this can be of importance in some cases. There is no clear idea on how to combine pushover analysis with actions at every nonlinear step that account for the vertical ground motion.

f) Structural capacity and seismic demand are considered independent in the current method. This is incorrect, as the inelastic structural response is load-path dependent and the structural capacity is always associated with the seismic demand.

2.5 ALTERNATE PUSHOVER ANALYSIS PROCEDURES

As discussed in the previous Section, pushover analysis lacks many important features of nonlinear dynamic analysis and it will never be a substitute for nonlinear dynamic analysis as the most accurate tool for structural analysis and assessment. Nevertheless, several possible developments can considerably improve the efficiency of the method. There are several attempts available in the literature to overcome the limitations of this analysis. These include the use of alternative lateral load patterns, use of higher mode properties and use of adaptive procedures. This Section presents some selected alternative procedures of pushover analysis.

2.5.1 Modal Pushover Analysis

Modal Pushover Analysis (MPA), developed by Chopra and Goel (2002)[12], is an improved procedure to calculate target displacement. This procedure is developed based on the differential equations governing the response of a multi-storey building subjected to an earthquake ground motion with acceleration, $\ddot{u}_g(t)$:

$$[m]\{\ddot{u}\}+[c]\{\dot{u}\}+[k]\{u\}=-[m]\{1\}\ddot{u}_g(t) \qquad (2.7)$$

where $\{u\}$ is the floor displacements relative to the ground, $[m]$, $[c]$, and $[k]$ are the mass, classical damping, and lateral stiffness matrices of the system.

The right side of Eq. 2.7 can be interpreted as the effective earthquake force vector:

$$\{P_{eff}(t)\} = -[m]\{1\}\ddot{u}_g(t) \tag{2.8}$$

Thus, the height-wise distribution of these forces can be defined by {*s*}= [*m*]{1}□ and their time variation by $\ddot{u}_g(t)$. This force distribution can be expanded as a combination of modal contributions $\{s_n\}$:

$$\{s\} = \sum_{n=1}^{N}\{s_n\} = \sum_{n=1}^{N}\Gamma_n[m]\{\phi_n\} \tag{2.9}$$

where { φ_n } is the *n*th mode of the structure and *N* is the number of modes to be considered. The modal pushover analysis method recommends to carryout pushover analysis separately for first few modes (satisfying response spectrum analysis rule) using the load pattern as given in Eq. 2.9. By utilizing the orthogonality property and decoupling of modes the solution of the differential equation (Eq. 2.7) can be written as:

$$\{u_n(t)\} = \{\phi_n\}q_n(t) = \Gamma_n\{\phi_n\}D_n(t) \tag{2.10}$$

where *q* (*t*) *n* is the modal coordinate, Γ_n is modal participation factor of the n^{th} mode and *D*(*t*) *n* is governed by the equation of motion for a SDOF system, with *n*th mode natural frequency ω_n and damping ratio ξ_n , subjected to $\ddot{u}_g(t)$:

$$\ddot{D}_n + 2\xi_n\omega_n\dot{D}_n + \omega_n^2 D_n = -\ddot{u}_g(t) \tag{2.11}$$

Now, the displacement at the roof due to n^{th} mode can be expressed as:

$$u_{n,roof}(t) = \Gamma_n\phi_{n,roof}D_n(t) \tag{2.12}$$

Where $\varphi_{n,roof}$ is the value of the n^{th} mode shape at roof level.

The peak value of the roof displacement due to n^{th} mode can be expressed as:

$$u_{no,roof} = \Gamma_n\phi_{n,roof}D_n \tag{2.13}$$

where D_n, the peak value of $D_n(t)$, can be determined by solving Eq. 2.11 or from the inelastic response spectrum. $u_{no,roof}$, is the target displacement of the building at roof due to n^{th} mode. The peak modal responses from all the modes considered are combined according to appropriate modal combination rule (such as SRSS, CQC, etc.).

This application of modal combination rules to inelastic systems obviously lacks a theoretical basis. However, it seems reasonable because it provides results for elastic buildings that are identical to the well-known RSA procedure. The lateral force distribution (Eq. 2.9) and the target displacement (Eq. 2.13) suggested for modal pushover analysis possesses two properties: (1) it keeps the invariant distribution of forces and (2) it provides the exact modal response for elastic systems.

The steps in the MPA procedure to estimate target displacement of a multi-storeyed building are summarised below.

i. Compute the natural frequencies (ω_n) and modes shapes $\{\square_n\}$ for linear elastic vibration of the building.

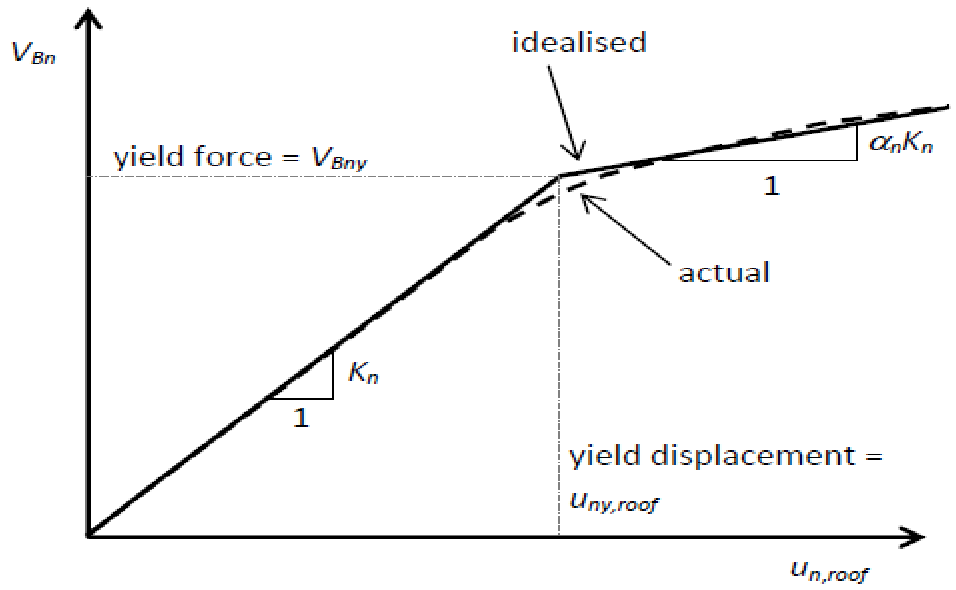
ii. For the n^{th} mode, develop the base shear versus roof displacement curve (pushover curve) for force distribution, $\Gamma_n[m]\{\square_n\}$ or just $[m]\{\square_n\}$.

iii. Idealise the pushover curve as a bilinear curve (Fig. 2.6). Convert the idealised base shear versus roof displacement curve of the multi-storeyed building to force-displacement relation for n^{th} mode inelastic equivalent SDOF system using the following relations:

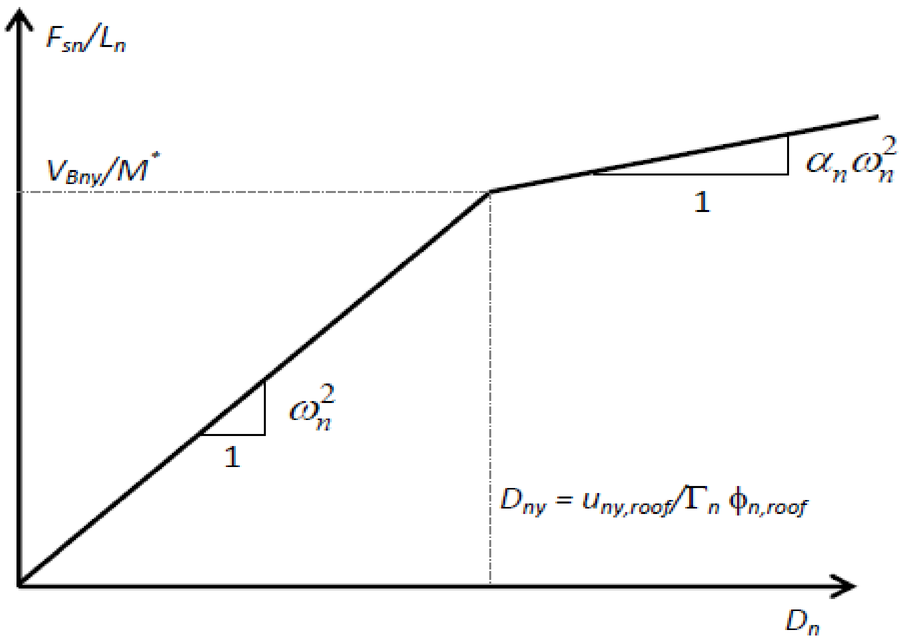
$$F_{sn} / L_n = V_{Bn} / M_n^* \text{ and } D_n = u_{n,roof} / \Gamma_n \phi_{n,roof}$$

where F_{sn} and D_n are the force and displacement for equivalent SDOF system corresponding to n^{th} mode. V_{Bn} and $u_{n,roof}$ are base shear and roof displacement obtained from pushover analysis with n^{th} mode shape as lateral load pattern. The purpose of this step is to obtain the properties of n^{th} mode equivalent inelastic SDOF system.

iv. Compute the peak deformation (D_n) of n^{th} mode inelastic equivalent SDOF system defined in the previous step, either from inelastic design spectrum or from the empirical equations.



(a) Idealised pushover curve



(b) $F_{sn}/L_n - D_n$ relationship

Fig. 2.6: Properties of the n^{th} mode inelastic SDOF system from the pushover curve[12].

Recent research shows that this procedure is capable of analysing buildings with plan asymmetry [35] and some forms of vertical irregularity [14]. However, a recent paper concludes that the scope of the applicability of multimode pushover analysis is not very wide and should be used with caution when analysing a particular category of buildings. A new modal combination rule (factored modal combination) to estimate the load profile for pushover analysis [36]. This combination is found to work for frames with vertical irregularities (soft ground story and vertical mass irregularity)

2.6 SUMMARY

This chapter presents a detailed literature review of seismic performance of setback buildings.

The chapter also presents the pushover analysis procedure, its limitations and recent improvements to this procedure. The research papers on setback buildings conclude that the displacement demand is dependent on the geometrical configuration of frame and concentrated in the neighbourhood of the setbacks for setback structures.

The higher modes significantly contribute to the response quantities of structure. Also conventional pushover analysis seems to be underestimating the response quantities in the upper floors of the irregular frames.

Pushover analysis as explained in the FEMA 356 is primarily meant for regular buildings with dominant fundamental mod participation. There are many alternative approaches of pushover analysis reported in the literature to make it applicable for different categories of irregular buildings. These comprise (i) *modal pushover analysis* [11], (ii) *modified modal pushover analysis* [14], *upper bound pushover analysis* [21] and (iv) *adaptive pushover analysis*, etc.

There is an effort by project ATC 55 to improve the current displacement coefficient method and capacity spectrum method. However, none of these alternative methods and the improved displacement coefficient and capacity spectrum method has been tested for setback buildings.

From the above conclusions, it is clear that the evaluation of seismic demands for setback buildings is necessary to assess the seismic performance of setback buildings.

CHAPTER-3

BUILDING CONFIGURATION AND ITS MODELLING FOR ANALYSIS

3.1 INTRODUCTION

The study in this report is based on non-linear analysis of structural models representing vertically irregular multi-storey buildings with setback. First part presents summary of various parameters of the computational model, the basic assumptions and geometries of the buildings were considered for study. It is important to model the non-linear properties accurately in non-linear analysis.

Frame elements were modelled with inelastic flexural hinges with point plasticity model. Second part explains the properties of hinges, the assumptions are made and procedure for generation of properties.

3.2 BUILDING CONFIGURATION AND MATERIAL PROPERTIES DETAILS

The buildings are assumed which are regular in plan are selected with respect to variation in number of bays, number of storeys and basically three types of configurations with equal setbacks in upper floors. Description of building frames are given in tabular form with basic assumptions as follows:

- Height of each storey: 3m
- Length of each bay (centre to centre in both direction): 4m
- Building configuration:

Square Building is selected due to same dimension in both directions and to study the impact of an earthquake.

Rectangular building is selected to study the comparative effect with respect to square building due to change in dimension.

L- Type building is selected to study the effects of earthquake forces on a unsymmetrical building

Table 3.1. Type of building configuration with setback

Type-I(Model 1)	Type-II(Model 2)	Type-III(Model 3)
Square	Rectangular	L-type
6 bays X 6 bays (24m*24m)	4 bays X 9 bays (16m*36m)	X-leg:28m Y-leg:32m
Setback with 320 m^2 at 16^{th} floor	Setback with 320 m^2 at 16^{th} floor	Setback with 320 m^2 at 16^{th} floor

R.C.C building which is considered as Special Moment Resisting Frame (SMRF) because its detailing conforms to **IS: 13920**.

Various other details related to building frames and material used is summarised in tabular form in table 3.2:

Table.3.2 Building geometry and material properties

S.N.	Parameters of design	Mathematical value
1.	Height of storey (c/c)	3m
2.	Beam size	350mm*500mm
3.	Column size	500mm*500mm
4.	Unit weight of concrete(RCC)	25kN/m^3
5.	Unit weight (masonry walls)	20 kN/m^3
6.	Characteristic strength of concrete(beam)(f_{ck})	25MPa
7.	Characteristic strength of concrete(column)	30MPa
8.	Elastic modulus of masonry infilled walls(E_m)	5500MPa
9.	Elastic modulus of concrete(E_c)	$5000\sqrt{f_{ck}}$
10.	Thickness of slab	150mm
11.	Thickness of masonry wall(exterior)	230mm
12.	Thickness of masonry wall(interior)	230mm
13.	Poisson's ratio for concrete	0.20
14.	Poisson's ratio for masonry infilled wall	0.17

3.3 LOAD CALULATIONS:

3.3.1 Seismic Design Data

The designed seismic data for assumed SMRF building frames is shown in table 3.3:

Table 3.3. Seismic Design Data

S.N.	Parameters for design	Values
i.	Seismic zone	IV
ii.	Zone factor	0.24
iii.	Importance factor (I)	1
iv.	Response reduction factor (R)	5
v.	Type of soil	Medium(Type-II)
vi.	Damping ratio	5%
vii.	Type of frame	Special moment resisting frame

3.3.2 Gravity Load Considered For Design:

Dead load (IS875: part-1)

i. Dead load of beams and columns: As per unit weight of material and dimensions.
ii. Dead load on floor/roof slabs(flooring load): 1.2kN/m^2
iii. Dead load on periphery beams(Exterior wall load,230 mm thick):11.5kN/m
iv. Dead load on interior beams(Interior wall load,230 mm thick): 11.5kN/m

Live load (IS875: part-2)

i. Live load on floor/roof slab :3 kN/m^2 (Residential Building)

As per IS 1893:2016, clause 7.3.1, the percentage of live load considered for seismic load calculation is 25% if live load is less than 3 kN/m^2.

3.4 MODELLING OF FRAMES AND MASONRY INFILLED WALLS

Beams and columns are modelled as 2D frame elements. Column bases were considered fixed for all models in the study. The entire frame elements are modelled with non-linear properties.

Diaphragm is assigned at each floor level for the structural effect of in-plane stiffness of slab.

3.4.1 Types of Plan Of Different Buildings With Setback

The study is based on setback buildings with 3m storey heights and 4m bay width. Three types of building geometry were taken in this study. The geometrics of building represents equal amount of setback area in all the three models above 15th floor levels. Bays varying from 6 to 9 in X as well as Y direction with uniform bay width of 4m have been considered. Different building plans with setback at different height have been shown in figures mentioned below:

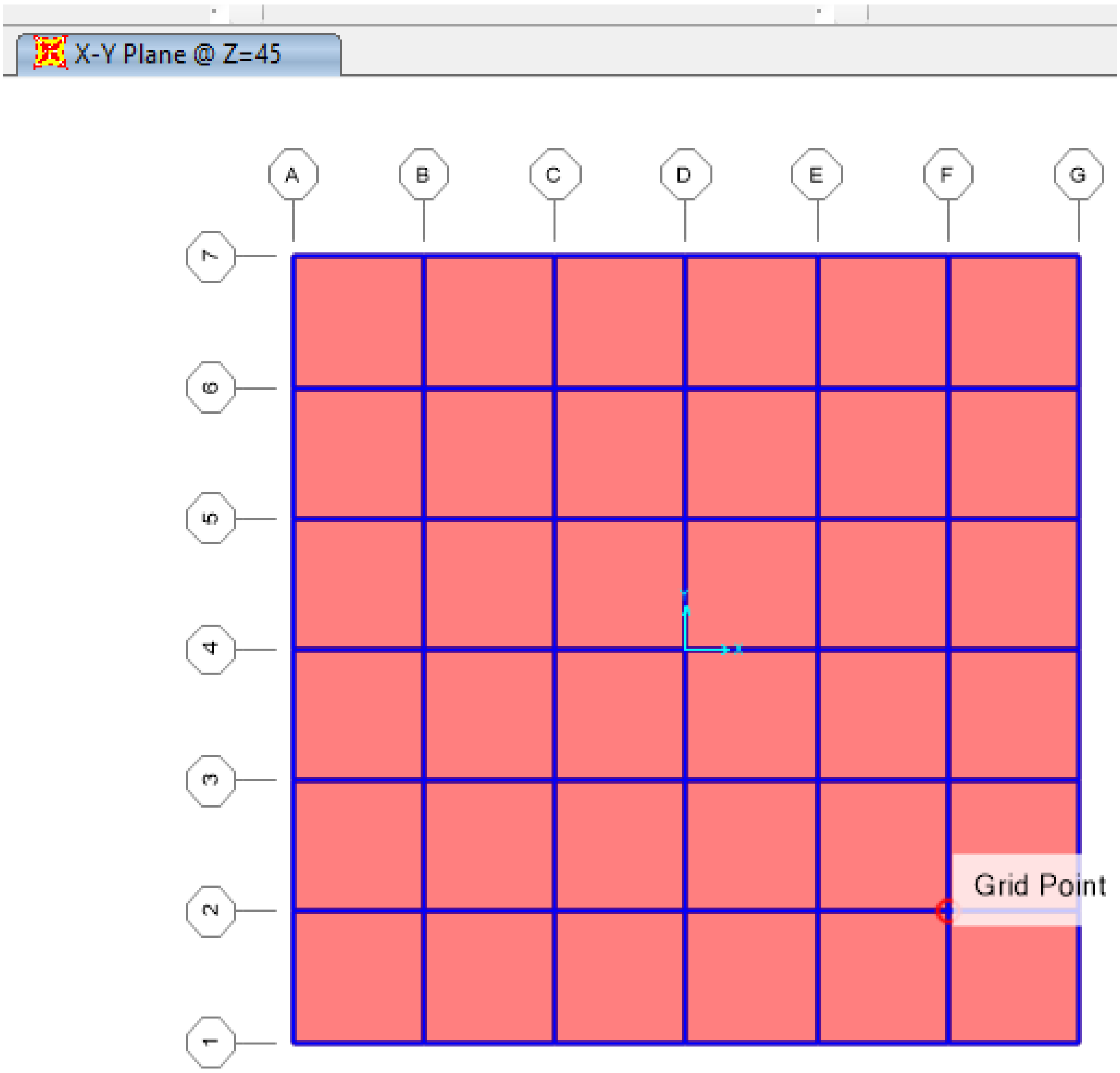


Fig 3.1(a) Plan of setback model 1 (square type) up to 45m height.

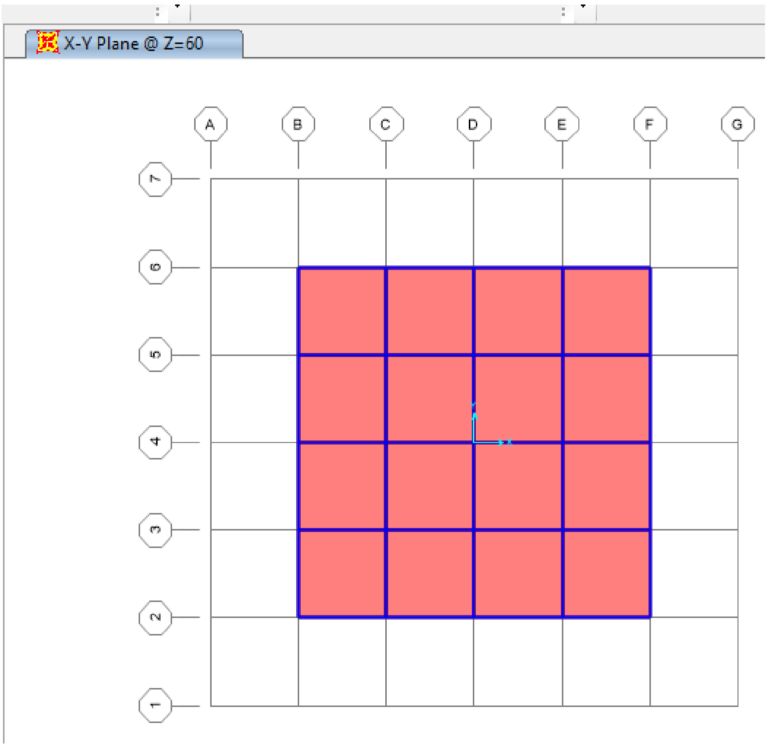


Fig 3.1(b) Plan of setback model 1 (square type) beyond 45m height.

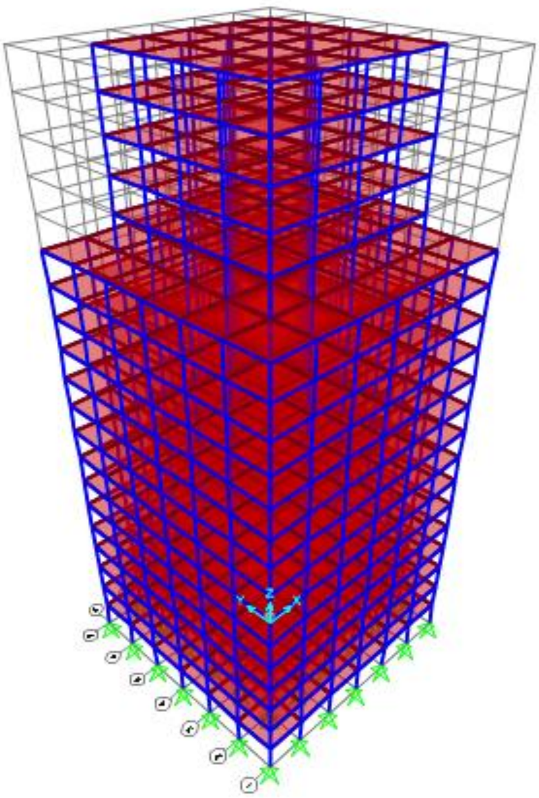


Fig 3.2 3-D view of model 1(square type)

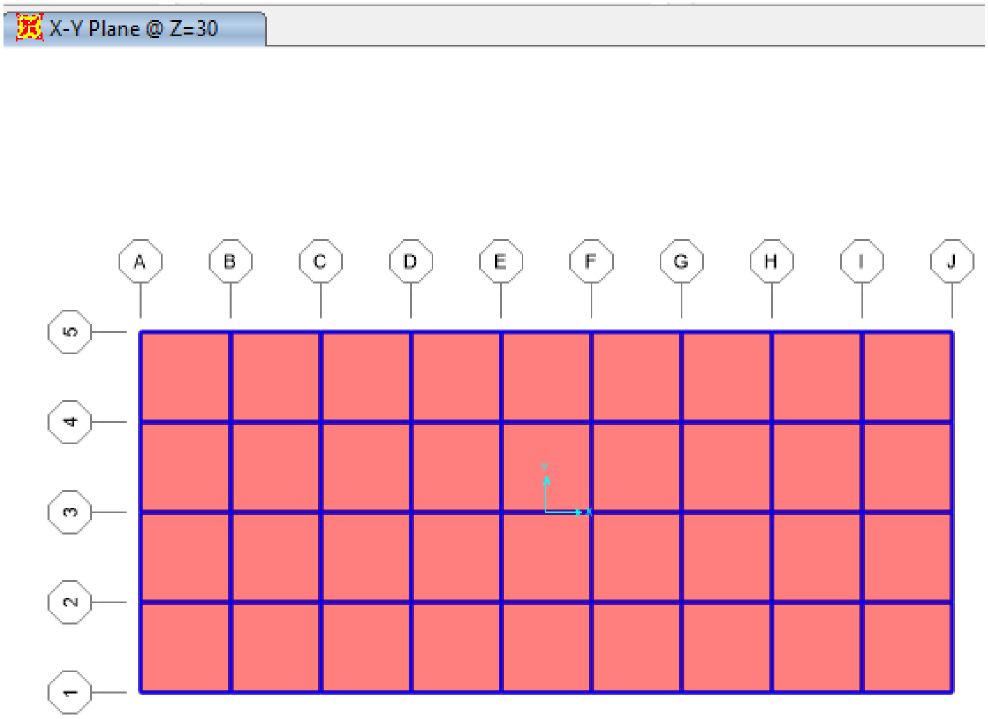


Fig 3.3 (a) plan of setback model 2(rectangular type) up to 45m height

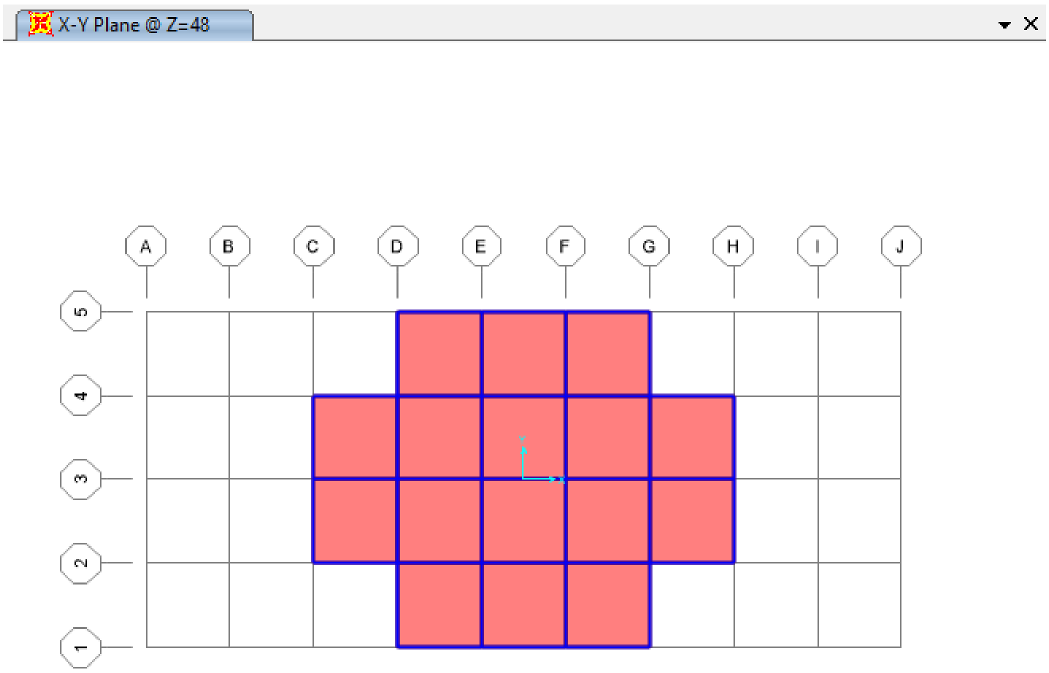


Fig 3.3 (b) plan of setback model 2(rectangular type) beyond 45m height

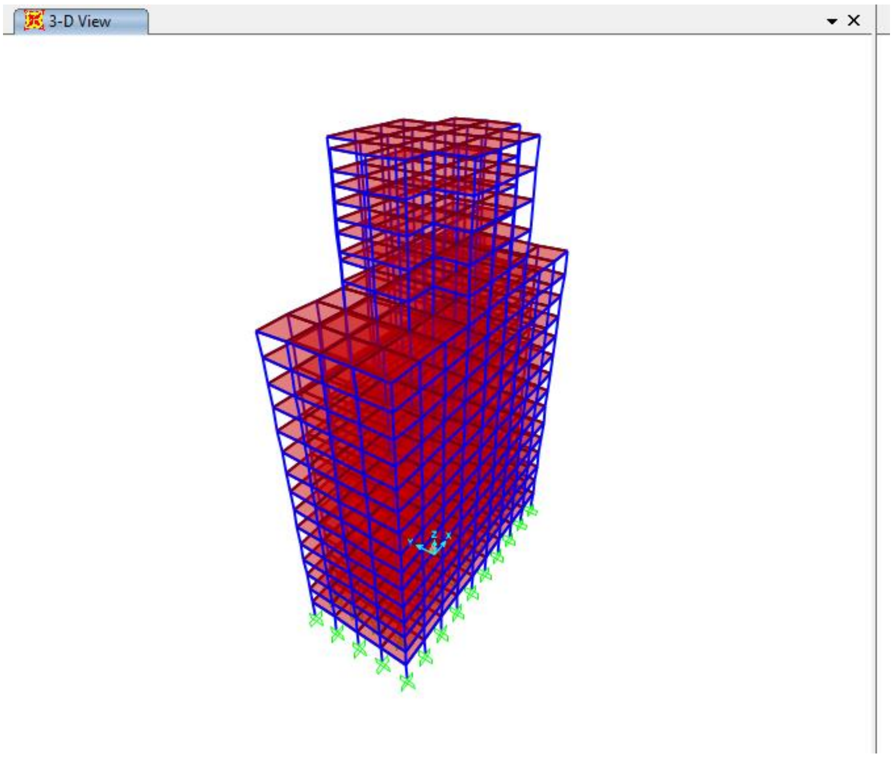


Fig 3.4 3-D view of model 2(rectangular type)

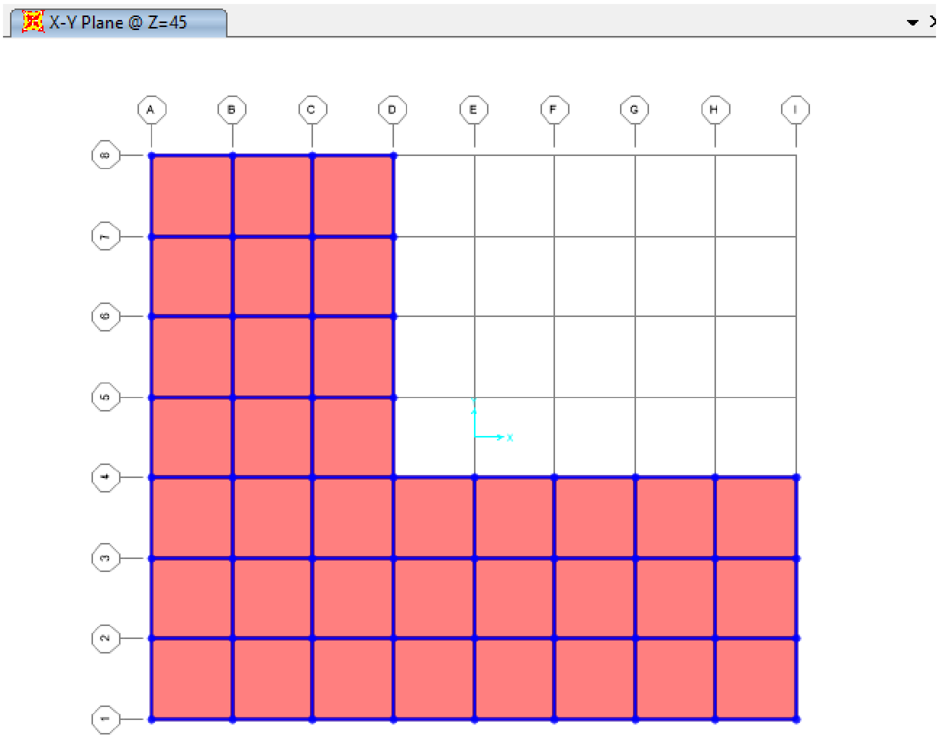


Fig 3.5 (a) plan of setback model 3 (L-type) up to 45 m height

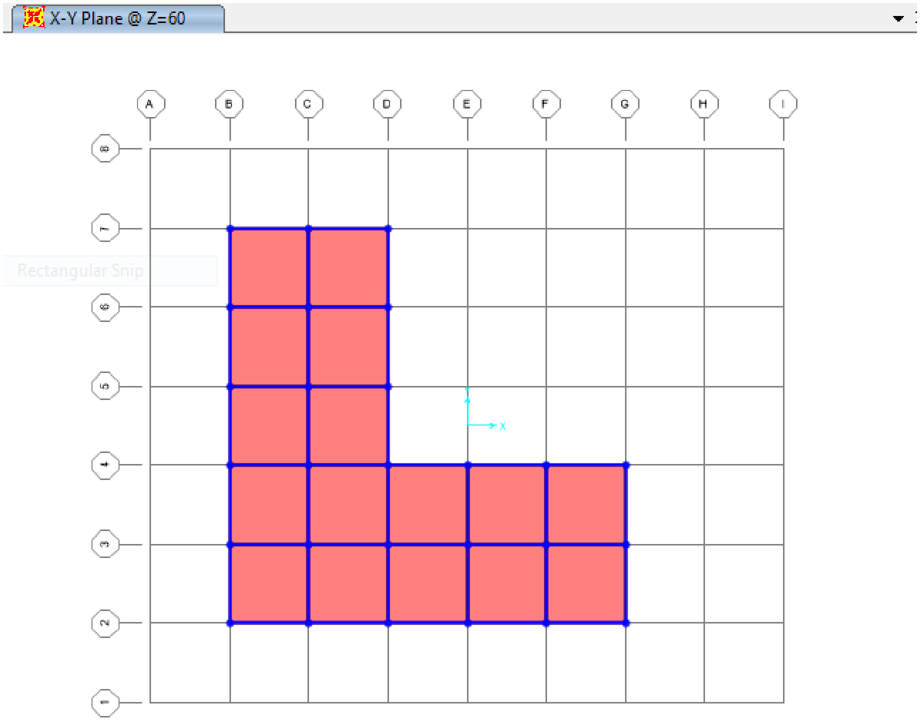


Fig 3.5 (b) plan of setback model 3(L-type) beyond 45m height

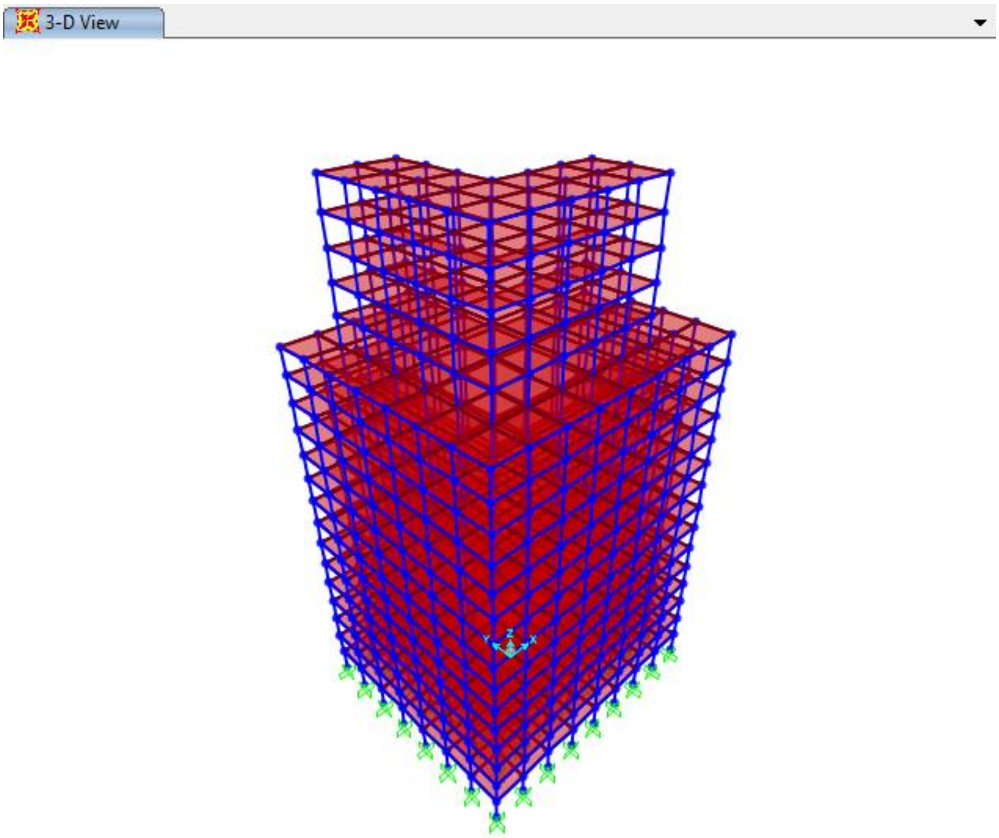


Fig 3.6 3-D view of model 3(L- type)

3.5 MODELLING OF STRUCTRAL ELEMENTS:

Pushover analysis is a non-linear static procedure in which structural load is increased incrementally with predefined pattern. ATC-40 and FEMA-356 documents describe the parameters. During analysis the yielding of frame members is also been described in FEMA-356. During analysis, the inelastic behaviour of structural elements two methods was governed as shown in fig. 3.7. First one is deformation controlled and second one is force controlled.

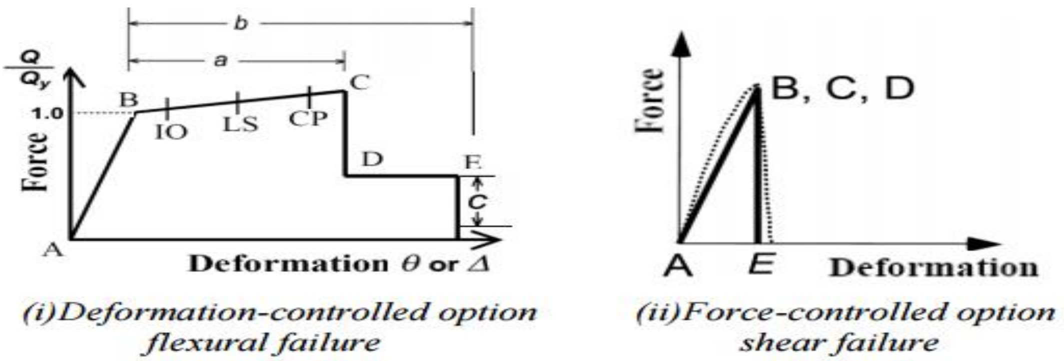


Fig 3.7 Force vs. deformation behaviour of hinges.

3.5.1 Performance level of columns and beams:

When a structure is analysed with three loading conditions (gravity, earthquake-x and earthquake-y), pushover curve is obtained. This is also base shear vs. deformation curve.

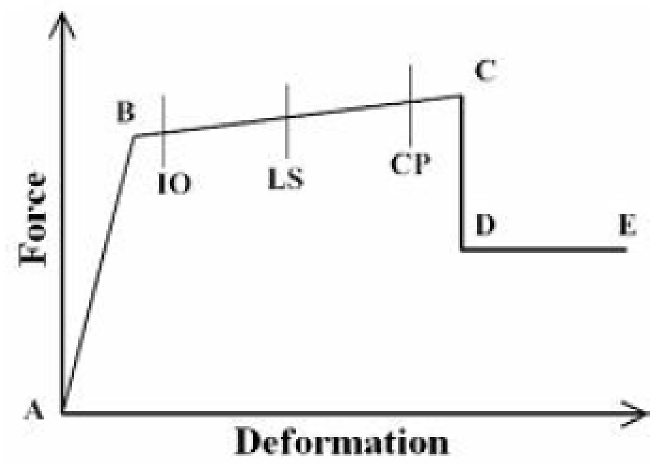


Fig 3.8 Force vs. deformation curve

Following key points have been drawn from the above curve:

- Point 'A' is the origin.

- Point ‘B’ is the yielding point. Up to this point no deformation takes place in the hinge. Beyond point ‘B’ only plastic deformation in hinge occurs.
- ‘C’ point represents ultimate capacity in pushover analysis.
- Residual strength is represented by point ‘D’ in the curve.
- ‘E’ is the point of total failure.

Points IO,LS and CP are used to describe the criteria for acceptance level of the plastic hinge formed near the joints (at ends of columns and beams), where IO- immediate occupancy, LS- life safety, CP- collapse prevention. The assigned value of each point depends up on the type of member and defined parameter in ATC-40 and FEMA-273 documents. Acceptance criteria values for columns and beams are mention in table 3.4 and table 3.5. Levels of structural performance are described in table 3.6.

Table 3.4 Modelling parameters of columns [16]

Conditions			Modeling Parameters[4]			Acceptance Criteria[4]				
						Plastic Rotation Angle, radians				
						Performance Level				
							Component Type			
			Plastic Rotation Angle, radians		Residual Strength Ratio		Primary		Secondary	
			a	b	c	IO	LS	CP	LS	CP
i. Columns controlled by flexure[1]										
$\frac{P}{A_g f'_c}$	Trans. Reinf.[2]	$\frac{V}{b_w d \sqrt{f'_c}}$								
≤ 0.1	C	≤ 3	0.02	0.03	0.2	0.005	0.015	0.02	0.02	0.03
≤ 0.1	C	≥ 6	0.016	0.024	0.2	0.005	0.012	0.016	0.016	0.024
≥ 0.4	C	≤ 3	0.015	0.025	0.2	0.003	0.012	0.015	0.018	0.025
≥ 0.4	C	≥ 6	0.012	0.02	0.2	0.003	0.01	0.012	0.013	0.02
≤ 0.1	NC	≤ 3	0.006	0.015	0.2	0.005	0.005	0.006	0.01	0.015
≤ 0.1	NC	≥ 6	0.005	0.012	0.2	0.005	0.004	0.005	0.008	0.012
≥ 0.4	NC	≤ 3	0.003	0.01	0.2	0.002	0.002	0.003	0.006	0.01
≥ 0.4	NC	≥ 6	0.002	0.008	0.2	0.002	0.002	0.002	0.005	0.008
ii. Columns controlled by shear[1, 3]										
All cases [5]			—	—	—	—	—	—	.0030	.0040
iii. Columns controlled by inadequate development or splicing along the clear height[1,3]										
Hoop spacing ≤ d/2			0.01	0.02	0.4	0.005	0.005	0.01	0.01	0.02
Hoop spacing > d/2			0.0	0.01	0.2	0.0	0.0	0.0	0.005	0.01
iv. Columns with axial loads exceeding $0.70P_o$[1, 3]										
Conforming hoops over the entire length			0.015	0.025	0.02	0.0	0.005	0.01	0.01	0.02
All other cases			0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

1. When more than one of the conditions i, ii, iii, and iv occurs for a given component, use the minimum appropriate numerical value from the table.
2. “C” and “NC” are abbreviations for conforming and nonconforming transverse reinforcement. A component is conforming if, within the flexural plastic hinge region, hoops are spaced at ≤ $d/3$, and if, for components of moderate and high ductility demand, the strength provided by the hoops (V_s) is at least three-fourths of the design shear. Otherwise, the component is considered nonconforming.
3. To qualify, columns must have transverse reinforcement consisting of hoops. Otherwise, actions shall be treated as force-controlled.
4. Linear interpolation between values listed in the table shall be permitted.

Table 3.5 Modelling parameters of beams [16].

Conditions			Modeling Parameters[3]			Acceptance Criteria[3]				
						Plastic Rotation Angle, radians				
						Performance Level				
							Component Type			
			Plastic Rotation Angle, radians		Residual Strength Ratio		Primary		Secondary	
Conditions			a	b	c	IO	LS	CP	LS	CP
i. Beams controlled by flexure[1]										
$\frac{\rho - \rho'}{\rho_{bal}}$	Trans. Reinf.[2]	$\frac{V}{b_w d \sqrt{f'_c}}$								
≤ 0.0	C	≤ 3	0.025	0.05	0.2	0.010	0.02	0.025	0.02	0.05
≤ 0.0	C	≥ 6	0.02	0.04	0.2	0.005	0.01	0.02	0.02	0.04
≥ 0.5	C	≤ 3	0.02	0.03	0.2	0.005	0.01	0.02	0.02	0.03
≥ 0.5	C	≥ 6	0.015	0.02	0.2	0.005	0.005	0.015	0.015	0.02
≤ 0.0	NC	≤ 3	0.02	0.03	0.2	0.005	0.01	0.02	0.02	0.03
≤ 0.0	NC	≥ 6	0.01	0.015	0.2	0.0015	0.005	0.01	0.01	0.015
≥ 0.5	NC	≤ 3	0.01	0.015	0.2	0.005	0.01	0.01	0.01	0.015
≥ 0.5	NC	≥ 6	0.005	0.01	0.2	0.0015	0.005	0.005	0.005	0.01
ii. Beams controlled by shear[1]										
Stirrup spacing ≤ d/2			0.0030	0.02	0.2	0.0015	0.0020	0.0030	0.01	0.02
Stirrup spacing > d/2			0.0030	0.01	0.2	0.0015	0.0020	0.0030	0.005	0.01
iii. Beams controlled by inadequate development or splicing along the span[1]										
Stirrup spacing ≤ d/2			0.0030	0.02	0.0	0.0015	0.0020	0.0030	0.01	0.02
Stirrup spacing > d/2			0.0030	0.01	0.0	0.0015	0.0020	0.0030	0.005	0.01
iv. Beams controlled by inadequate embedment into beam-column joint[1]										
			0.015	0.03	0.2	0.01	0.01	0.015	0.02	0.03

1. When more than one of the conditions i, ii, iii, and iv occurs for a given component, use the minimum appropriate numerical value from the table.
2. "C" and "NC" are abbreviations for conforming and nonconforming transverse reinforcement. A component is conforming if, within the flexural plastic hinge region, hoops are spaced at ≤ $d/3$, and if, for components of moderate and high ductility demand, the strength provided by the hoops (V_s) is at least three-fourths of the design shear. Otherwise, the component is considered nonconforming.
3. Linear interpolation between values listed in the table shall be permitted.

Table 3.6 Performance levels of concrete frames [16]

Elements	Type	Structural Performance Levels		
		Collapse Prevention	Life Safety	Immediate Occupancy
Concrete Frames	Primary	Extensive cracking and hinge formation in ductile elements. Limited cracking and/or splice failure in some nonductile columns. Severe damage in short columns.	Extensive damage to beams. Spalling of cover and shear cracking (< 1/8" width) for ductile columns. Minor spalling in nonductile columns. Joint cracks < 1/8" wide.	Minor hairline cracking. Limited yielding possible at a few locations. No crushing (strains below 0.003).
	Secondary	Extensive spalling in columns (limited shortening) and beams. Severe joint damage. Some reinforcing buckled.	Extensive cracking and hinge formation in ductile elements. Limited cracking and/or splice failure in some nonductile columns. Severe damage in short columns.	Minor spalling in a few places in ductile columns and beams. Flexural cracking in beams and columns. Shear cracking in joints < 1/16" width.
	Drift[2]	4% transient or permanent	2% transient; 1% permanent	1% transient; negligible permanent

3.5.2 Non-Linear Modelling of Beams and Columns

It is essential to model load deformation curve of all elements in pushover analysis.

The columns and beams are modelled as frame elements. Diaphragm action is assigned to slabs in modelling. It is essential to model the load versus deformation curve, as deformations likely go beyond elastic range.

It is necessary to incorporate the non-linear behaviour to the load versus deformation property of hinge connected to the member. A moment versus rotations hinge is assigned to a beam. To model the expected shear failure of a section, shear force versus shear deformation curve is plotted. Column is assigned with shear and flexible hinges.

3.6 BEHAVIOUR PARAMETER OF BUILDING:

In force based seismic design procedure, R is the factor for force reduction used to reduce the linear elastic response spectra to inelastic response spectra. For structures to remain linearly static, it is designed for seismic force less than the expected under strong ground motion,

$$R=V_e/V_d$$

R= response reduction factor (empirical) which counts overstrength, damping and ductility in the structural system at greater displacements after initial yield and approachable to displacement at ultimate load in structure.

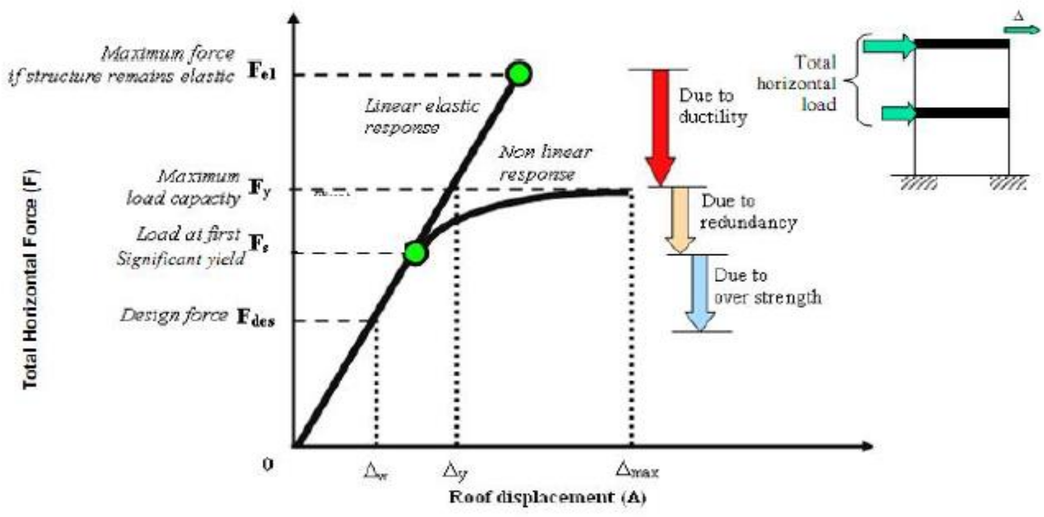


Fig 3.9 Concept of response reduction factor

3.6.1 Response reduction factor formulation:

ATC-19 describes R which consists of three factors

$$R= R_S.R\mu.R_R$$

Where R_S represents overstrength, which is the ratio of maximum base shear at yield (V_y) to design base shear (V_d)

$R\mu$ represents ductility factor, which is the ratio of base shear at elastic response (V_e) to base shear yield (V_y)

R_R is the factor of redundancy and depends upon the number of vertical frames participation in seismic resistance.

3.6.2 Overstrength factor:

After the structure reaches if ultimate strength and deformation capacity, the strength beyond designed strength is known as overstrength.

Overstrength factor (Ω)= Apparent strength /design strength

$$\Omega = V_u/V_d$$

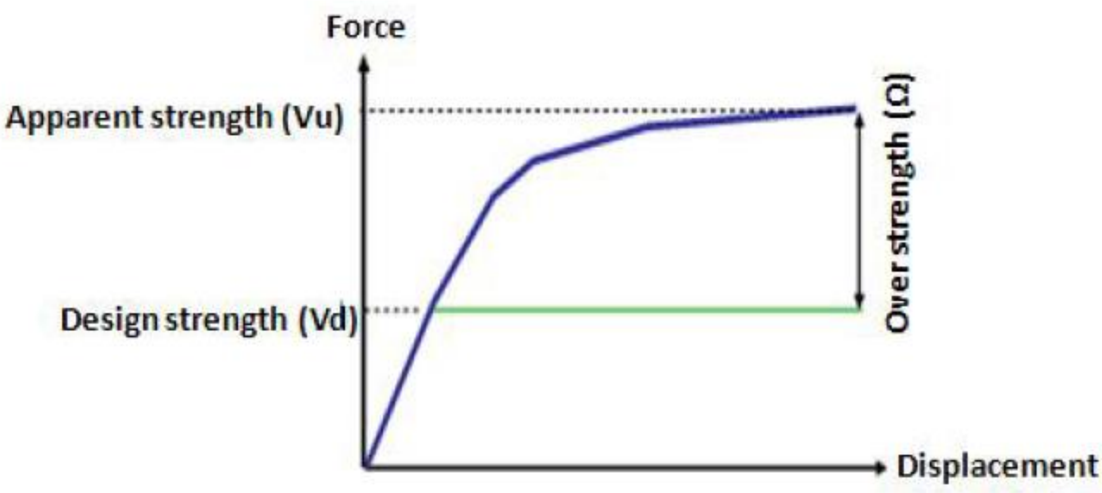


Fig 3.10 Force vs. displacement curve for over strength

3.7 SUMMARY:

In this chapter, the basic modelling for linear and non-linear analysis are presented in details. Description of the building configurations considered in present study is also done.

CHAPTER-4

RESULTS AND DICUSSIONS

4.1 NON-LINEAR STATIC ANALYSIS RESULTS

Non –linear static analysis (pushover analysis) has been done to all the three type of buildings (square, rectangular and L-type) with equal setback provided in each model.

The analysis is performed in sap2000 (version 19).

Hinge formation at different steps of the considered setback buildings has been represented in tabular form given below:

Table 4.1 Hinge formation during pushover steps of setback model 1 (square type)

Pushover Capacity Curve

File View Edit Format-Filter-Sort Select Options

Units: As Noted

Filter:

LoadCase Text	Step Unitless	Displacement	BaseForce KN	AtoB Unitless	BtoIO Unitless	IOtoLS Unitless	LStoCP Unitless	CPtoC Unitless	CtoD Unitless	DtoE Unitless
push-X	235	0.28202	19060.169	4640	0	0	0	0	0	0
push-X	236	0.28322	19141.277	4640	0	0	0	0	0	0
push-X	237	0.28442	19222.385	4640	0	0	0	0	0	0
push-X	238	0.28562	19303.492	4640	0	0	0	0	0	0
push-X	239	0.28682	19384.595	4637	3	0	0	0	0	0
push-X	240	0.28802	19465.524	4635	5	0	0	0	0	0
push-X	241	0.28922	19546.335	4633	7	0	0	0	0	0
push-X	242	0.29042	19627.022	4632	8	0	0	0	0	0
push-X	243	0.29162	19707.626	4629	11	0	0	0	0	0
push-X	244	0.29282	19788.097	4628	12	0	0	0	0	0
push-X	245	0.29402	19868.49	4625	15	0	0	0	0	0
push-X	246	0.29522	19948.714	4624	16	0	0	0	0	0
push-X	247	0.29642	20028.895	4621	19	0	0	0	0	0
push-X	248	0.29762	20108.806	4621	19	0	0	0	0	0
push-X	249	0.29882	20188.72	4617	23	0	0	0	0	0

Record: << < 1 > >> of 392 Done

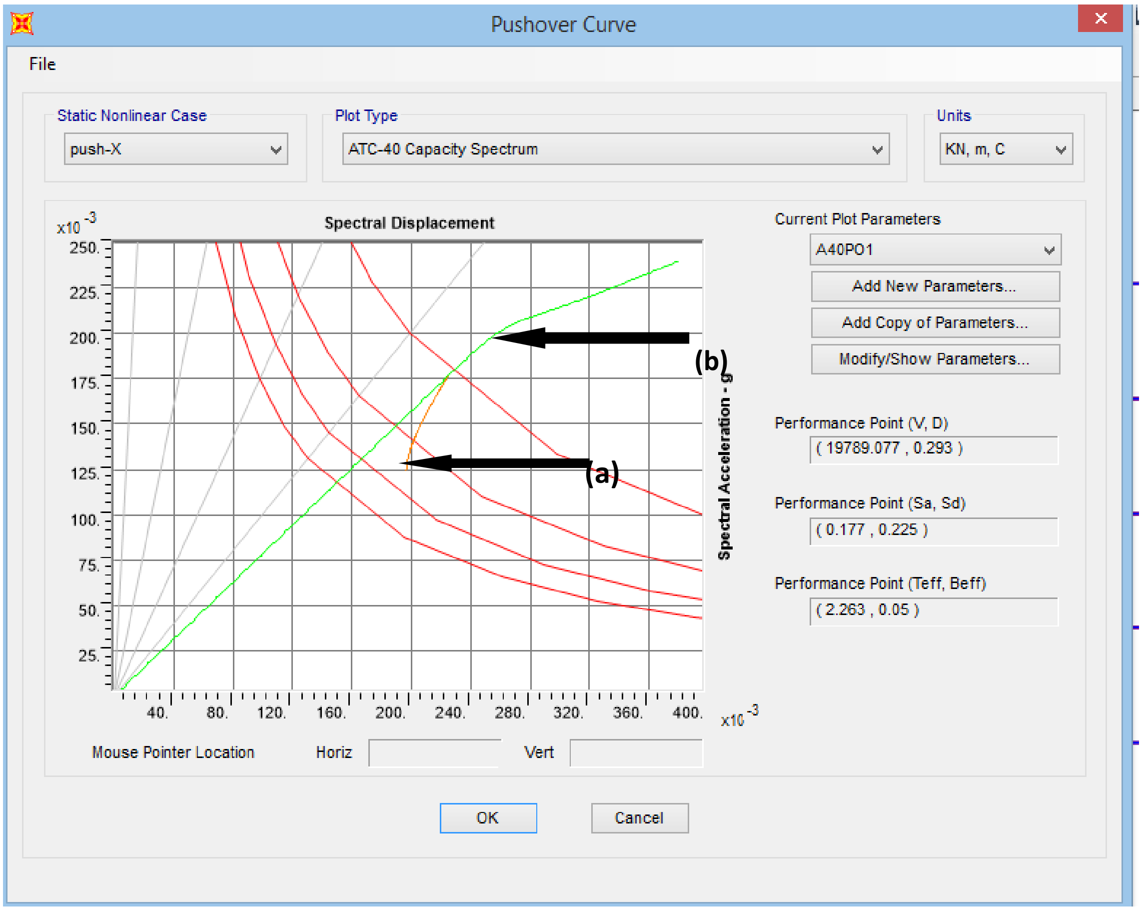


Fig.4.1 Pushover curve of setback model 1 (square type)

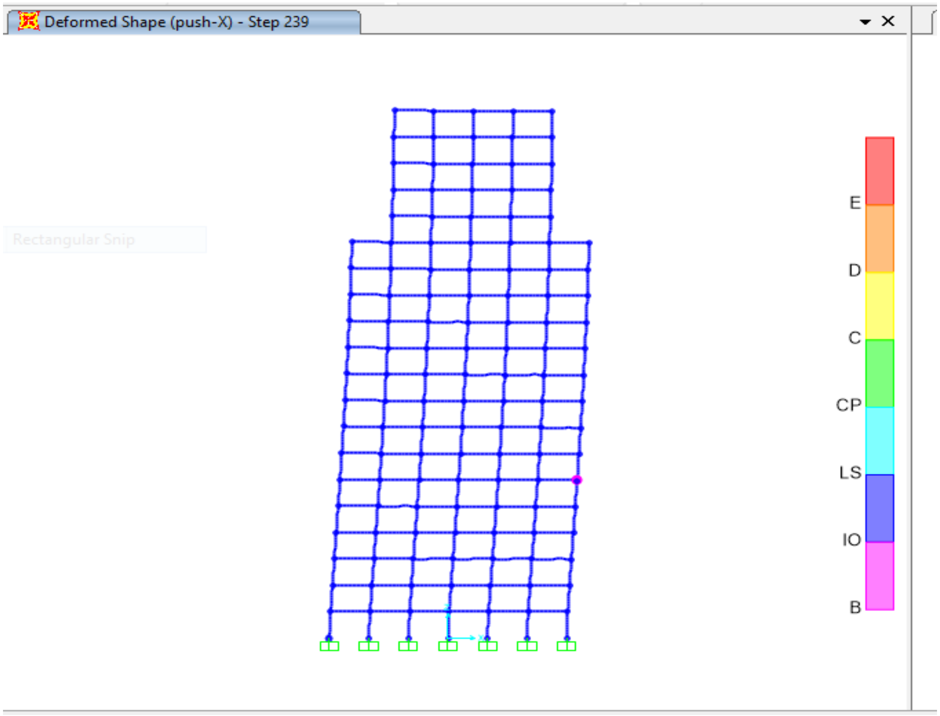


Fig.4.2 (a) Hinge formation during pushover of setback model 1 (square type) at step-239

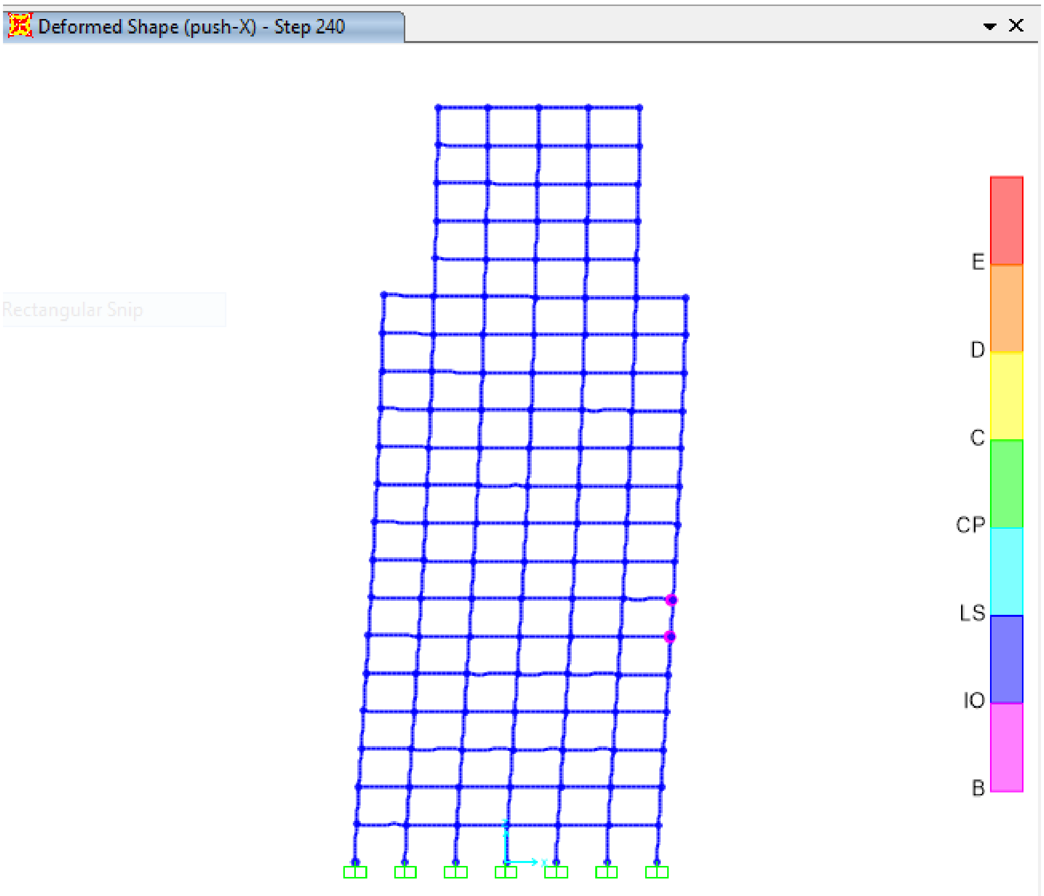
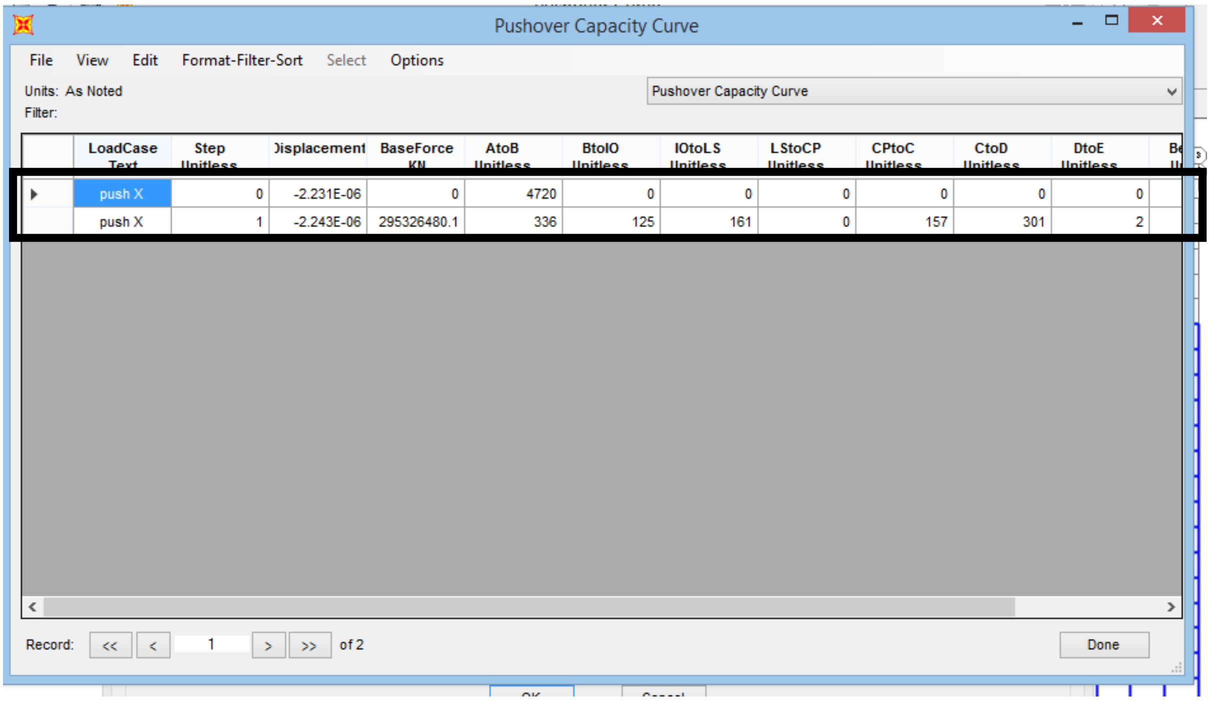


Fig.4.2 (b) Hinge formation during pushover of setback model 1 (square type) at step-240

Discussions drawn from the above table and the figures of hinge formation of setback building model -1 (square type):

- From the above pushover curve (fig 4.1) the following key points are:
  i) Curve ‘a’ indicates demand spectrum and curve ‘b’ indicates capacity spectrum, where ‘a’ and ‘b’ intersects is known as performance point.
- It is observed that base shear at performance point is 19789.077 kN with corresponding displacement 0.293 m.
- Plastic hinge formation in this model starts from step-239.
- Performance points remain between step-244 and step-245 of pushover in x-direction.
- Plastic hinges formed at step-244 are 12 in number.
- Since we have designed the structure for linear analysis and check the performance level of the structure, it is found that around 0.26% plastic hinges formed at performance point are within immediate occupancy level.

Table 4.2 (a) Hinge formation during pushover steps (in X-direction) of setback model 2 (rectangular type)



Pushover Capacity Curve

File View Edit Format-Filter-Sort Select Options

Units: As Noted

Filter:

LoadCase Text	Step Unitless	Displacement	BaseForce KN	AtoB Unitless	BtoIO Unitless	IOtoLS Unitless	LStoCP Unitless	CPtoC Unitless	CtoD Unitless	DtoE Unitless
push X	0	-2.231E-06	0	4720	0	0	0	0	0	0
push X	1	-2.243E-06	295326480.1	336	125	161	0	157	301	2

Record: 1 of 2

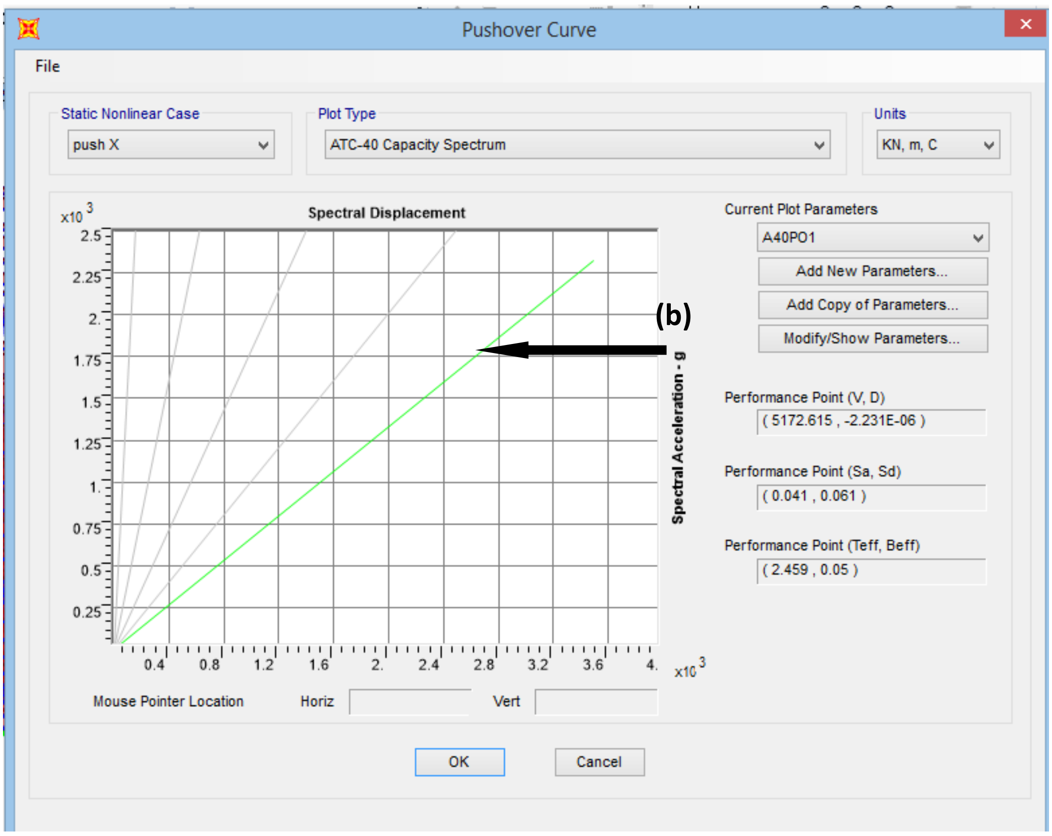


Fig.4.3 Pushover curve (X- direction) of setback model 2 (rectangular type)

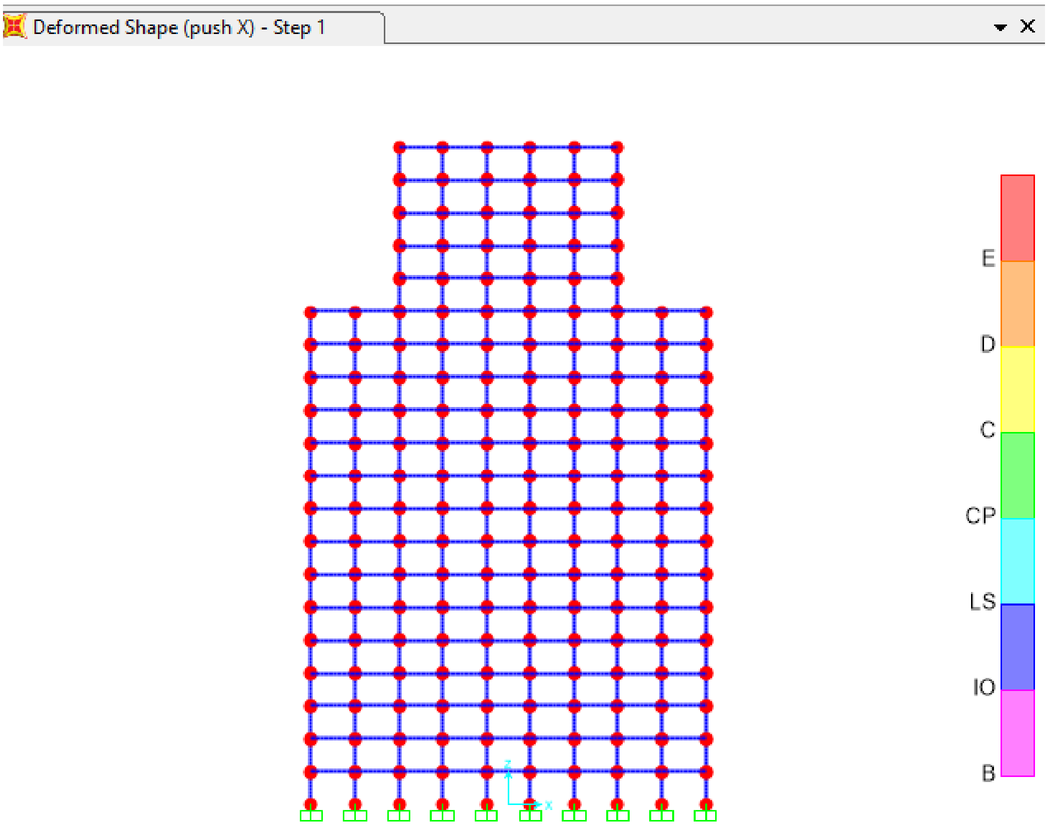


Fig.4.4 Hinge formation during pushover in x-direction of setback model 2 (rectangular type) at step-1

Table 4.3 Hinge formation during pushover (in Y-direction) of setback model 2 (rectangular type)

Pushover Capacity Curve

Units: As Noted

LoadCase Text	Step Unitless	Displacement	BaseForce KN	AtoB Unitless	BtoIO Unitless	IOtoLS Unitless	LStoCP Unitless	CPtoC Unitless	CtoD Unitless	DtoE Unitless
push- Y	18	0.043203	2522.196	4720	0	0	0	0	0	0
push- Y	19	0.045603	2662.319	4720	0	0	0	0	0	0
push- Y	20	0.048003	2802.442	4720	0	0	0	0	0	0
push- Y	21	0.050403	2942.565	4720	0	0	0	0	0	0
push- Y	22	0.052803	3082.687	4720	0	0	0	0	0	0
push- Y	23	0.055203	3222.806	4720	0	0	0	0	0	0
push- Y	24	0.05704	3330.058	4717	3	0	0	0	0	0
push- Y	25	0.060099	3504.822	4692	28	0	0	0	0	0
push- Y	26	0.063053	3665.479	4665	55	0	0	0	0	0
push- Y	27	0.065481	3791.161	4636	84	0	0	0	0	0
push- Y	28	0.068568	3942.626	4598	122	0	0	0	0	0
push- Y	29	0.070986	4054.88	4570	150	0	0	0	0	0
push- Y	30	0.073474	4165.38	4538	182	0	0	0	0	0
push- Y	31	0.07604	4273.741	4504	216	0	0	0	0	0
push- Y	32	0.078938	4391.069	4465	255	0	0	0	0	0

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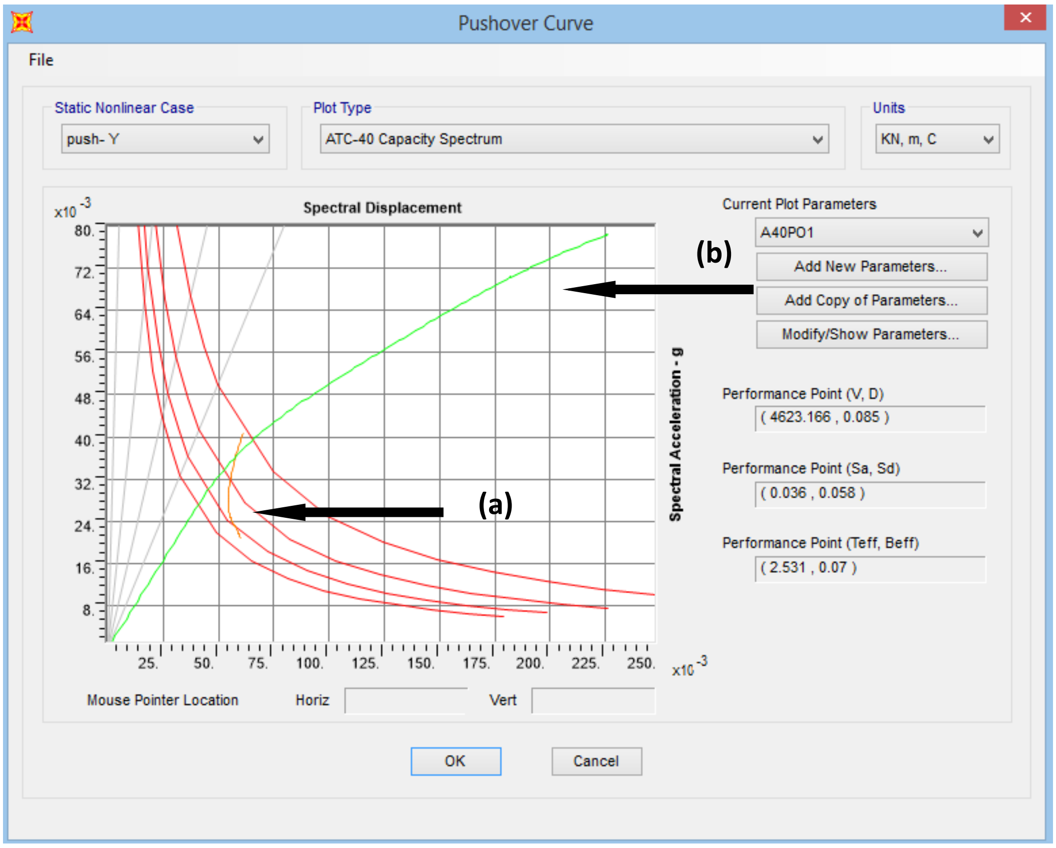
Fig.4.5 Pushover curve (Y- direction) of setback model 2 (rectangular type)



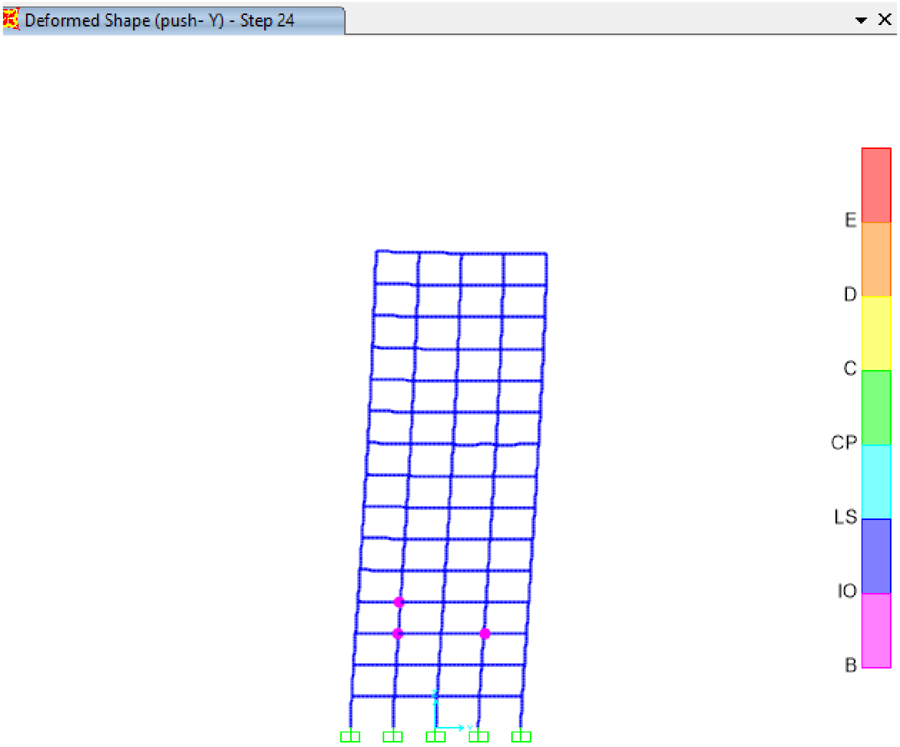


Fig.4.6 (a) Hinge formation during pushover in y-direction of setback model 2 (rectangular type) at step-24

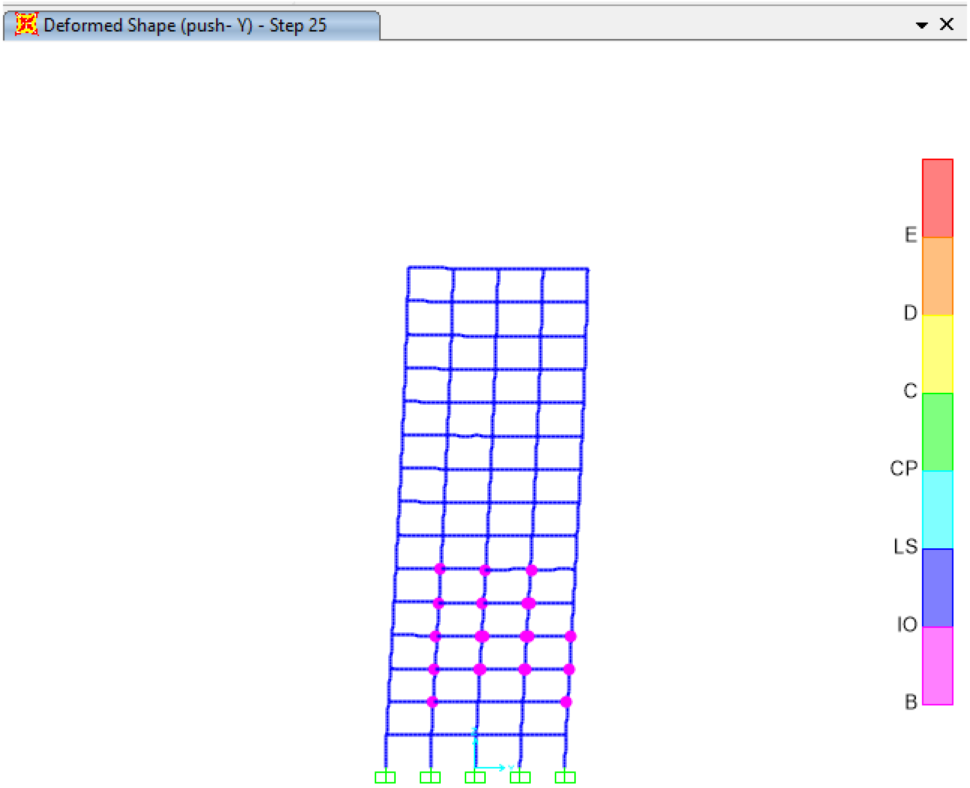
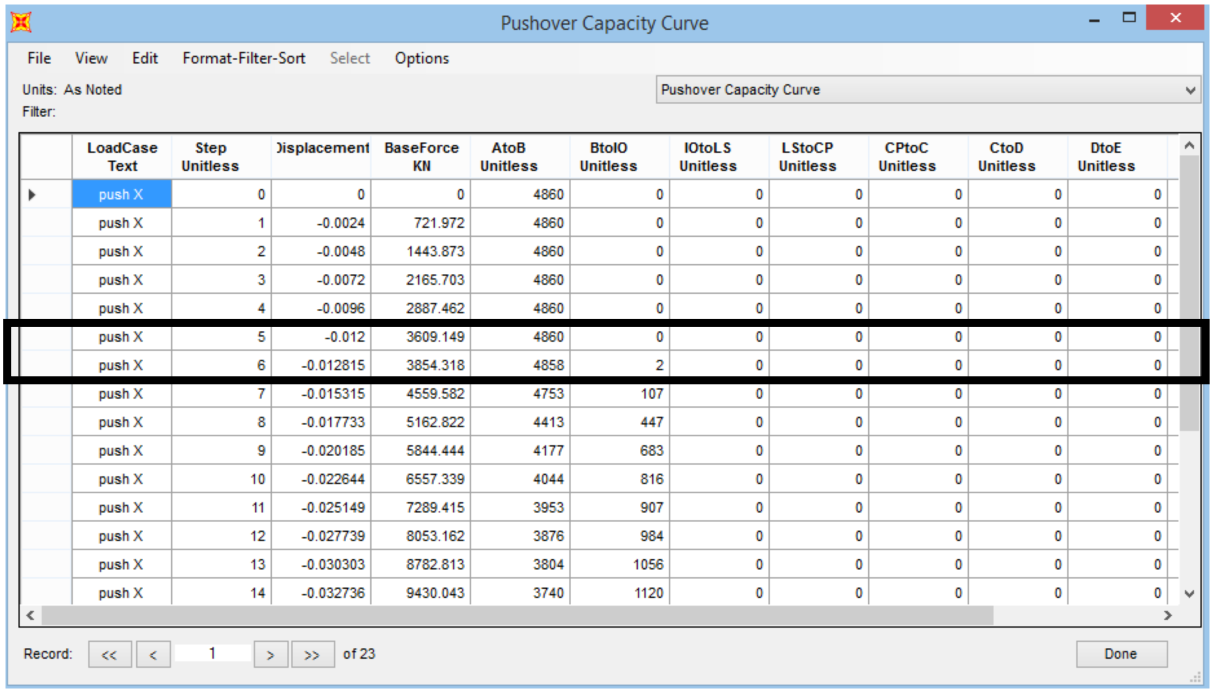


Fig.4.6 (b) Hinge formation during pushover in y-direction of setback model 2 (rectangular type) at step-25

Discussion drawn from the hinge formation during pushover in both x and y direction of setback building model-2 (rectangular type) summarised as:

- From the above pushover curve (fig 4.3 and fig 4.5) the following key points are:
  i) Curve ‘a’ indicates demand spectrum and curve ‘b’ indicates capacity spectrum, where ‘a’ and ‘b’ intersects is known as performance point.
- Base shear in x-direction at performance point is 5172.615 kN with respect to displacement -2.231E-06.
- Pushover in y-direction performance point comes nearly between step-34 and step - 35 with number of plastic hinges formed are 308 and 334 respectively.
- Percentage of plastic hinges formed remains with 6.5%
- Base shear in y-direction at performance point is 4623.166 kN with displacement 0.085m.
- The maximum storey drift remains within 0.1% at performance point in both x and y direction.

Table 4.4 Hinge formation during pushover steps (in X-direction) of setback model 3 (L- type)



Pushover Capacity Curve

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Units: As Noted

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Pushover Capacity Curve

LoadCase Text	Step Unitless	)isplacement	BaseForce KN	AtoB Unitless	BtoIO Unitless	IOtoLS Unitless	LStoCP Unitless	CPtoC Unitless	CtoD Unitless	DtoE Unitless
push X	0	0	0	4860	0	0	0	0	0	0
push X	1	-0.0024	721.972	4860	0	0	0	0	0	0
push X	2	-0.0048	1443.873	4860	0	0	0	0	0	0
push X	3	-0.0072	2165.703	4860	0	0	0	0	0	0
push X	4	-0.0096	2887.462	4860	0	0	0	0	0	0
push X	5	-0.012	3609.149	4860	0	0	0	0	0	0
push X	6	-0.012815	3854.318	4858	2	0	0	0	0	0
push X	7	-0.015315	4559.582	4753	107	0	0	0	0	0
push X	8	-0.017733	5162.822	4413	447	0	0	0	0	0
push X	9	-0.020185	5844.444	4177	683	0	0	0	0	0
push X	10	-0.022644	6557.339	4044	816	0	0	0	0	0
push X	11	-0.025149	7289.415	3953	907	0	0	0	0	0
push X	12	-0.027739	8053.162	3876	984	0	0	0	0	0
push X	13	-0.030303	8782.813	3804	1056	0	0	0	0	0
push X	14	-0.032736	9430.043	3740	1120	0	0	0	0	0

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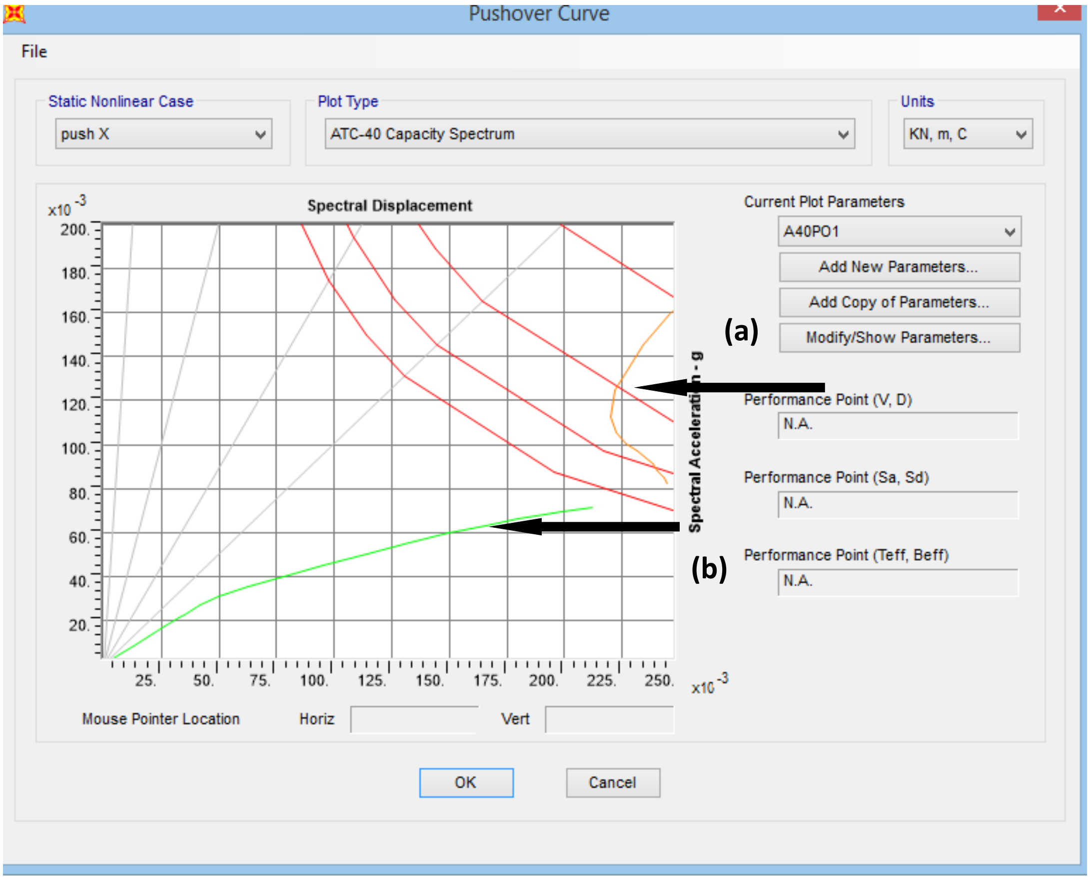

Fig.4.7 Pushover curve (X- direction) of setback model 3 (L- type)

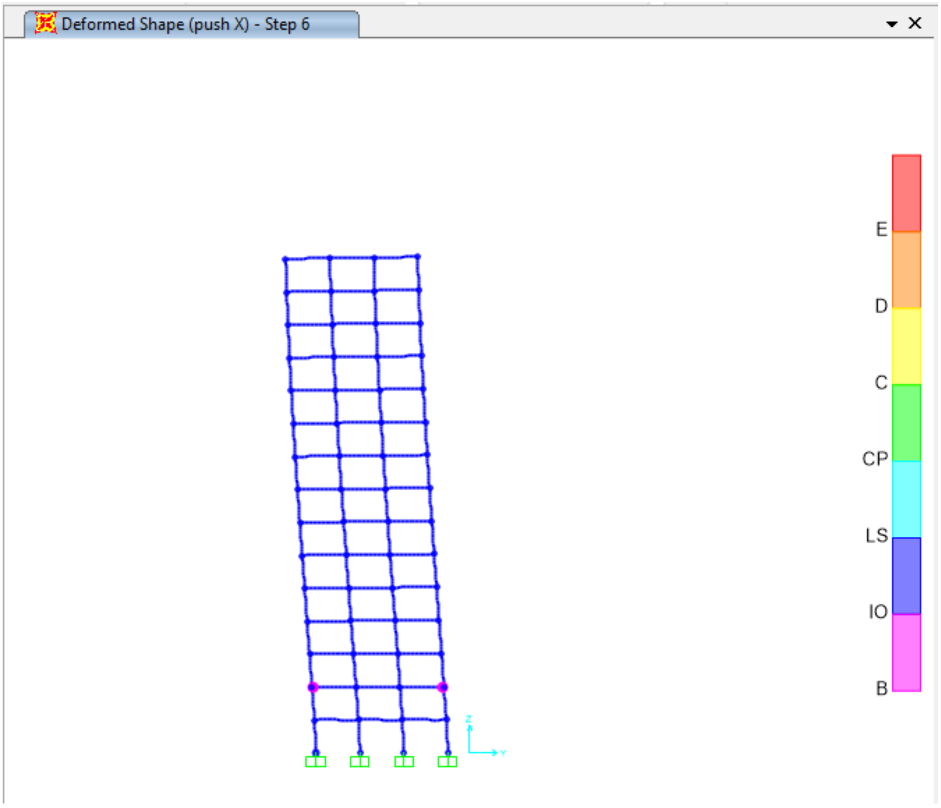

Fig.4.8 (a) Hinge formation during pushover in x-direction of setback model 3 (L- type) at step- 6

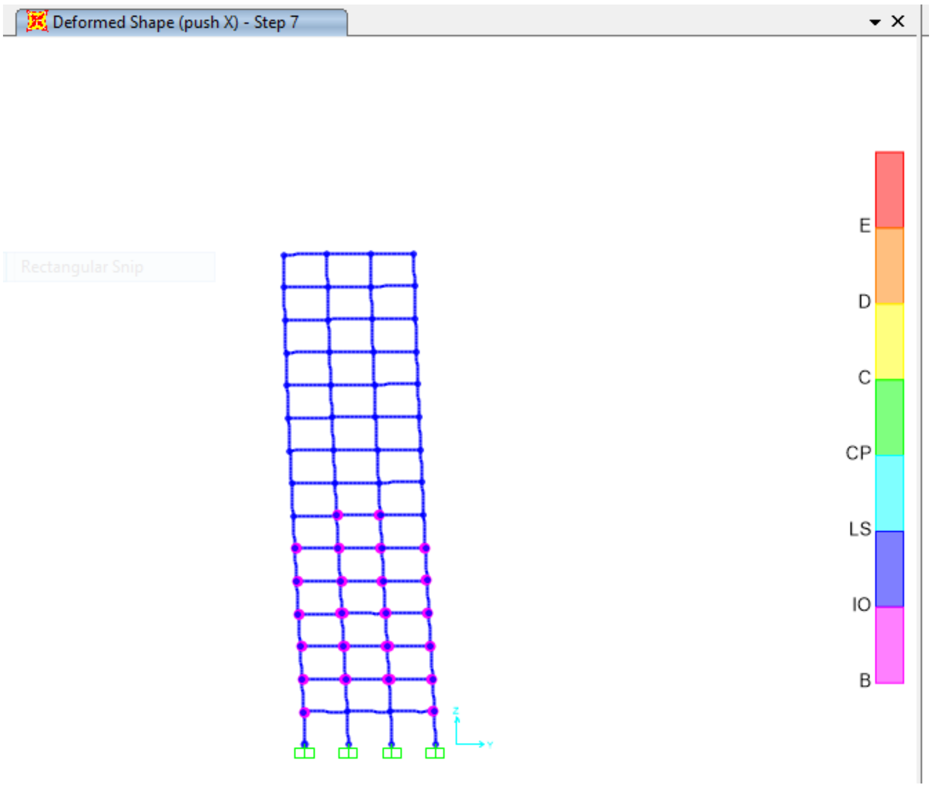

Fig.4.8 (b) Hinge formation during pushover in x-direction of setback model 3 (L- type) at step-7

Table 4.5 Hinge formation during pushover steps (in Y-direction) of setback model 3 (L- type)

its: As Noted
ter:

Pushover Capacity Curve

LoadCase Text	Step Unitless	Displacement	BaseForce KN	AtoB Unitless	BtoIO Unitless	IOtoLS Unitless	LStoCP Unitless	CPtoC Unitless	CtoD Unitless	DtoE Unitless	BeyondE Unitless	Total Unitless
push Y	0	0.000738	0	4851	9	0	0	0	0	0	0	4860
push Y	1	-0.001662	172.722	4851	9	0	0	0	0	0	0	4860
push Y	2	-0.004062	345.444	4851	9	0	0	0	0	0	0	4860
push Y	3	-0.005819	471.853	4850	10	0	0	0	0	0	0	4860
push Y	4	-0.010073	777.871	4848	12	0	0	0	0	0	0	4860
push Y	5	-0.014694	1109.93	4845	15	0	0	0	0	0	0	4860
push Y	6	-0.018172	1359.606	4840	20	0	0	0	0	0	0	4860
push Y	7	-0.022152	1644.57	4835	25	0	0	0	0	0	0	4860
push Y	8	-0.025693	1897.389	4827	33	0	0	0	0	0	0	4860
push Y	9	-0.029084	2138.298	4819	41	0	0	0	0	0	0	4860
push Y	10	-0.031679	2321.537	4803	57	0	0	0	0	0	0	4860
push Y	11	-0.035161	2564.891	4790	70	0	0	0	0	0	0	4860
push Y	12	-0.037643	2736.749	4777	83	0	0	0	0	0	0	4860
push Y	13	-0.040502	2933.229	4763	97	0	0	0	0	0	0	4860
push Y	14	-0.042935	3099.355	4760	100	0	0	0	0	0	0	4860
push Y	15	-0.045333	3262.292	4748	112	0	0	0	0	0	0	4860
push Y	16	-0.04824	3458.412	4733	127	0	0	0	0	0	0	4860
push Y	17	-0.050818	3630.79	4707	153	0	0	0	0	0	0	4860
push Y	18	-0.053579	3810.924	4671	189	0	0	0	0	0	0	4860
push Y	19	-0.056471	3988.804	4620	240	0	0	0	0	0	0	4860
push Y	20	-0.059111	4142.229	4580	280	0	0	0	0	0	0	4860
push Y	21	-0.061671	4285.059	4550	310	0	0	0	0	0	0	4860
push Y	22	-0.064454	4434.356	4512	348	0	0	0	0	0	0	4860
push Y	23	-0.067611	4597.405	4478	382	0	0	0	0	0	0	4860
push Y	24	-0.070668	4750.777	4434	426	0	0	0	0	0	0	4860

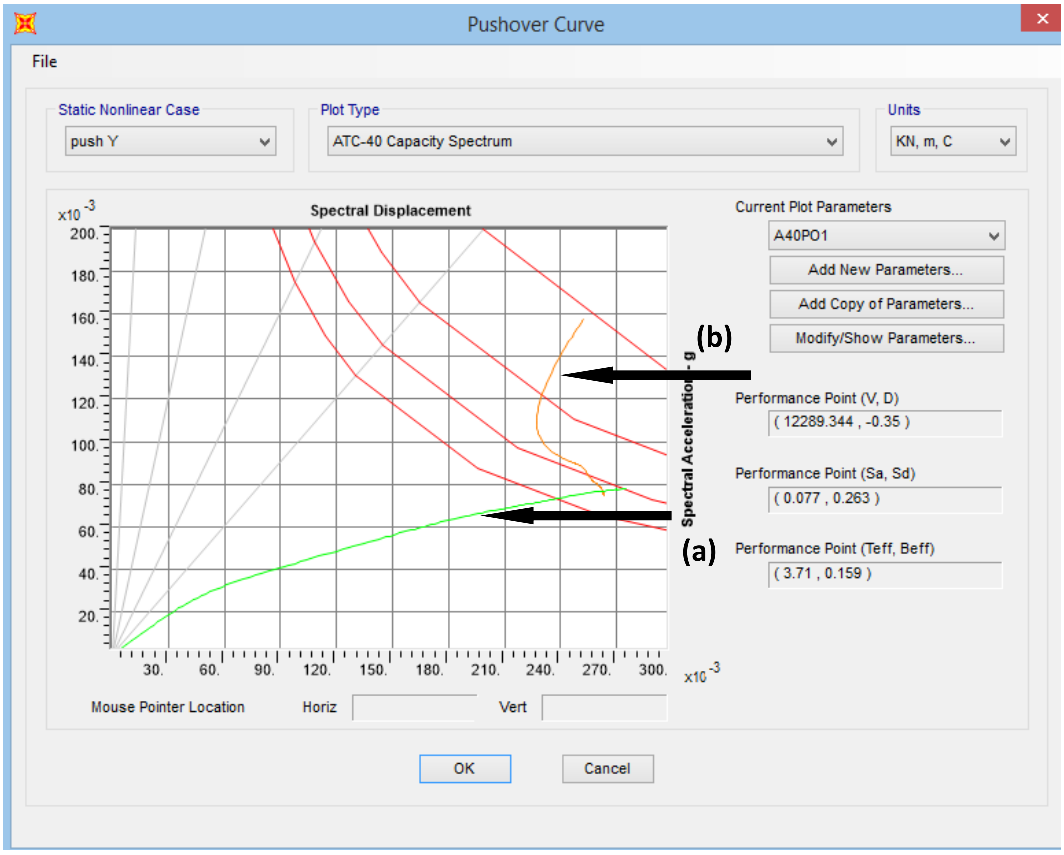

Fig.4.9 Pushover curve (Y- direction) of setback model 3 (L- type)

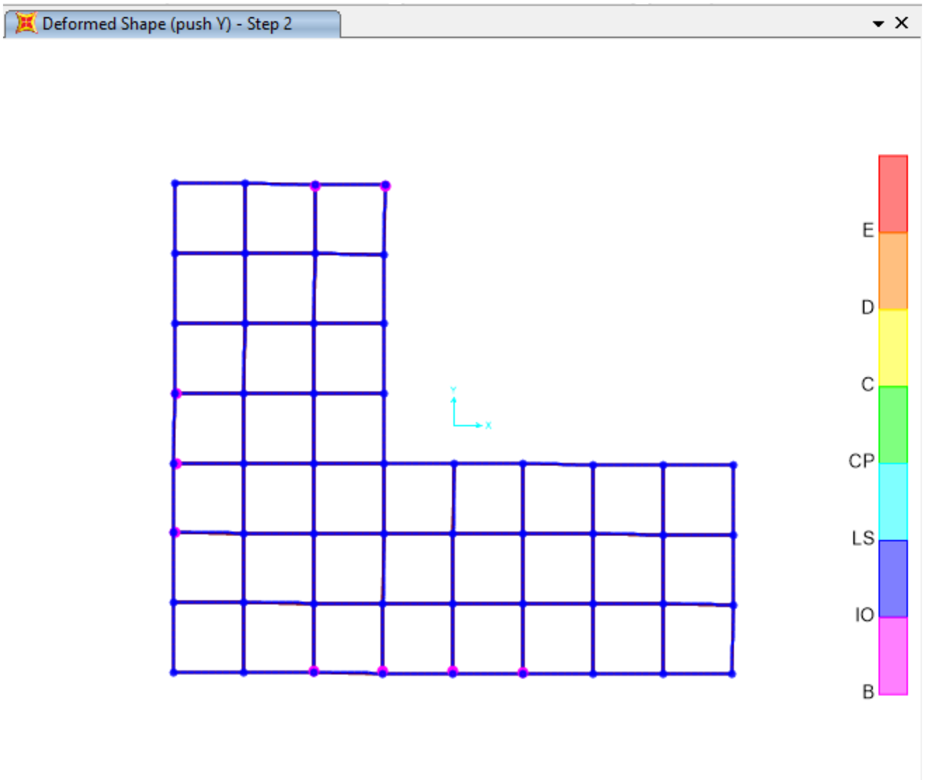

Fig.4.10 (a) Hinge formation during pushover in y-direction of setback model 3 (L- type) at step-2

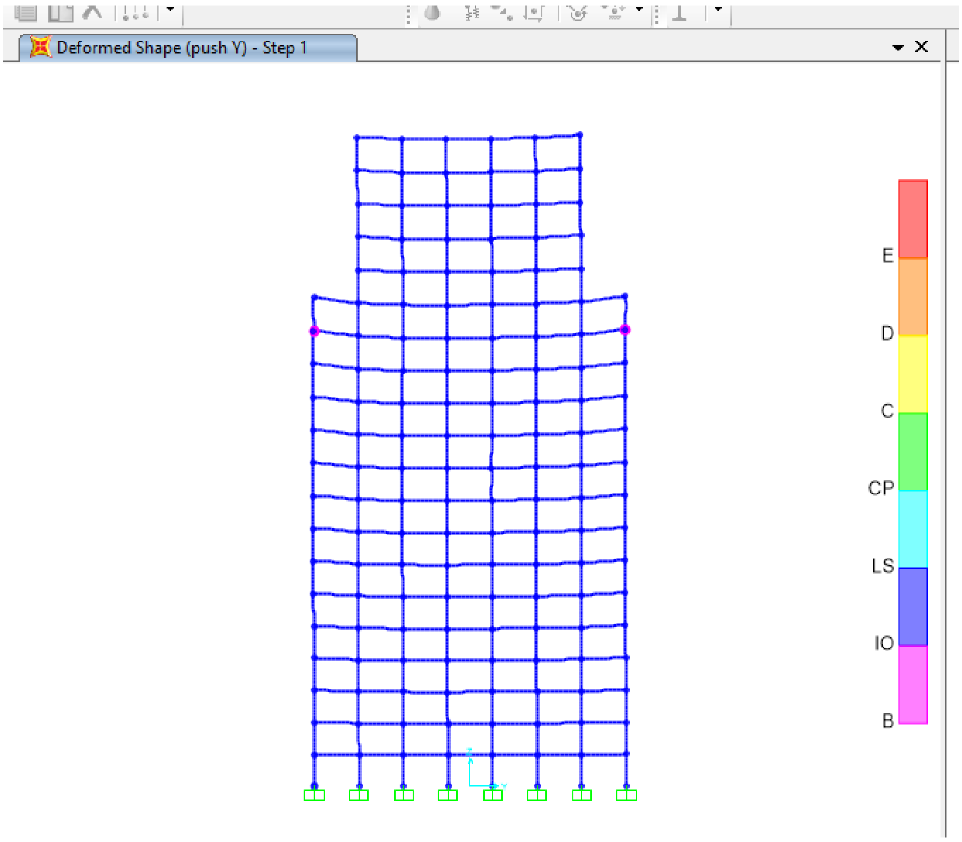

Fig.4.10 (b) Hinge formation during pushover in y-direction of setback model 3 (L- type) at step-1

Discussions drawn from the hinge formation during pushover in both x and y direction of setback building model-3 (L- type) summarised as follows:

- From the above pushover curve (fig 4.7 and fig 4.9) the following key points are:
  i) Curve ‘a’ indicates demand spectrum and curve ‘b’ indicates capacity spectrum, where ‘a’ and ‘b’ intersects is known as performance point.
- Performance point not applicable to this model.
- The corresponding base shear is found to be not applicable.
- Plastic hinge formation push in x direction at step 5 is 0 whereas in step 6 it is 02. Maximum number of members undergone inelastic deformation immediately in Step 7 is 107.
- In y-direction performance point is found at base shear 12289.344 kN with corresponding displacement -0.035m.
- Performance point lies between step1 and step 2 of pushover in y-direction.
- Number of plastic hinges formed in y-direction is 9 within performance point.
- The maximum storey drift remains within 0.6% at performance point in y direction.

CHAPTER 5

CONCLUSION

Based on the work presented in this report with equal plan area and equal setback following conclusions are drawn:

1. A detailed literature review on setback buildings conclude that the displacement demand is dependent on the geometrical configuration of frames.
2. The maximum base shear induced in the buildings is found to be more in Square type (model 1 in X- direction) setback building.
3. The base shear and corresponding displacement induced in the building within performance point is minimum in case of Rectangular - type of setback building (In Y Direction).
4. The Square - type setback building has maximum displacement within performance point.
5. From the comparison the maximum base shear at collapse occurs in Square-Type setback model (x- direction).
6. Number of plastic hinges formed within performance point is less in case of Square-type of setback building i.e., hinge formation as compared to other type of buildings.
7. It is observed that for all type of structural elements of outer periphery entered in plastic zone before internal elements due their farther placement.
8. In case of L-type setback building (In Y Direction) some hinges exceeds the limit of immediate occupancy without any performance point making it more susceptible to earthquake ground motion due to additional twisting effect.

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