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Performance Assessment of Mid rise Reinforced Concrete  
Frame with Shear Wall using pushover analysis

*A Thesis Submitted to the School of Graduate Studies of Addis Ababa University in Partial Fulfilment of the Requirements for the Degree of Masters of Science in Structural Engineering.*

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## **ABSTRACT**

It is difficult to model analytically the complex behaviour of reinforced concrete structure in its non-linear zone. This has led engineers in the past to heavily rely on empirical formulas which were derived from numerous experiments for the design and assessment of reinforced concrete members. It is also used to study the behaviour of reinforced concrete structure of force redistribution. This analysis of non-linear response of reinforced concrete structure is carried out in a routine fashion. It helps in the investigation of the behaviour of the structure under different loading conditions, its load deflection behaviour and crack pattern.

For reinforced concrete structures constructed in seismically active zone, the major cause of failure is ground excitation. This is true because of the unpredictable nature of the seismic excitation or because of proper structural design, detailing and construction. During seismic excitation it is possible to properly estimate the response of structures. For response determination of structures it is difficult to predict the real time response of structure using codes. Therefore, existing performance assessment must be done using a reliable and easily applicable analysis method.

As we know a major change on the expected peak ground acceleration (from 0.05g to 0.1g) shown in the revised Ethiopian building code (EBCS-EN-2014) for Addis Ababa. For the past decade large numbers of buildings are designed and constructed using the old building code. This change highly affects the performance of the structures. Hence, it is mandatory to assess the performance of existing structures using provisions of new code. This case study focuses on forty-sixty living apartames buildings as they are constructed in large scale all over the country and as they are areas where people are congregated. This study is done under the assumption that premature failure is not expected to happen.

This, study makes a comparative study on the effects of shear wall for different structural arrangement over different values of peak ground acceleration.

The three dimensional model for two type of building that is to say one regular and another irregular for G+7 building have been taken as case study where, all result found are three dimensional model result. This is chosen due to the fact that it will give the true picture of the behaviour of the selected building structures.

The analysis of pushover is done using ETABS2016 and SAP2000 (V18) and results has been compared on the displacement, story drift, base shear verses storey height. The performance points and pushover curve are plotted to check the input of new Ethiopian building code provision. The performance level is also computed to compare old and new Ethiopian code provision over the computation between regular and irregular structure over the presence and absence of shear wall.

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## LIST OF NOTATIONS

$c$  = viscous damping coefficient

$C_0$  = modification factor to relate the SDOF spectral displacement to MDOF roof displacement

$C_1$  = modification factor to related the expected maximum inelastic SDOF displacement divided by the elastic SDOF displacement

$C_2$  = modification factor to represent the effect of hysteresis shape on the maximum displacement response

$C_3$  = modification factor to represent increased displacement due to second-order effects

$C_R$  = ratio of peak deformations of inelastic and corresponding elastic SDOF systems for Systems with known yield-strength reduction factor.

$C_\mu$  = ratio of peak deformations of inelastic and corresponding elastic SDOF systems for systems with known yield-strength reduction factor.

$d_i$  = the lateral drift in story  $i$

$d_r$  = design inter-story drift, evaluated difference of the average lateral displacements at the top and bottom of the storey under consideration

$D_n$  = peak spectral roof displacement of the  $n^{\text{th}}$  mode SDF system

$E_D$  = energy dissipated through hysteretic behaviour

$E_S$  = strain energy at the maximum displacement

$g$  = acceleration of gravity

$h_i$  = height of  $i^{\text{th}}$  story

$k_i$  = elastic lateral stiffness of the building

$k_e$  = effective lateral stiffness of the building

$[K]$  = stiffness matrix

$K$  = stiffness of SDOF system

$[M]$  = mass matrix

$m$  = mass of SDOF system

$p_i$  = the portion of the total weight of the building

$R$  = ratio of inelastic strength demand to calculated yield strength coefficient

$R_y$  = yield-strength reduction factor  
 $S_a$  = spectral pseudo-acceleration  
 $S_d$  = spectral pseudo-displacement  
 $T_e$  = effective fundamental period  
 $T_o$  = characteristics period of the response spectrum  
 $T_a$  = period defined in Newmark-Hall smooth design spectrum  
 $T_b$  = period defined in Newmark-Hall smooth design spectrum  $\zeta$   
 $T_c$  = period separating acceleration –and velocity-sensitive regions  
 $T_c'$  = transition period in Newmark-Hall  $R_y$ - $\mu$ - $T_n$  relations  
 $T_n$  = elastic natural period of vibration  
 $u_m$  = peak roof of the  $n^{\text{th}}$  mode MDF system  
 $u_{rjn} = n^{\text{th}}$  mode roof displacement in the  $j$ -direction  
 $u$  = relative displacement between the mass and base of the structure  
 $\dot{U}$  = velocity of mass with respect to time  
 $\ddot{U}$  = acceleration of mass with respect to time  
 $V_{by}$  = base shear at yield at yield  
 $V_i$  = total calculated lateral shear force in the direction under consideration at story  $i$   
 $W$  = total dead load  
 $\zeta$  = viscous damping  
 $\delta$  = reference displacement of MDOF model  
 $\delta_t$  = target displacement of MDOF model  
 $\zeta_{eq}$  = equivalent damping value  
 $\zeta_{eff}$  = effective damping value  
 $\Omega$  = forcing frequency  
 $\kappa$  = damping modification  
 $\gamma$  = behaviour factor  
 $\theta_i$  = story drift sensitivity coefficient  
 $\theta_m$  = the maximum value of  $\theta_i$  for all stories  
 $\phi_n$  = mode shape factor  
 $\phi_m$  = amplitude of  $\phi_n$  at the roof in the direction of the selected pushover curve  
 $\phi_{rjn} = n^{\text{th}}$ -mode amplitude of mode shape at the roof level in the  $j$ -direction  
 $\Gamma_n$  = modal participation factor for the  $n^{\text{th}}$  mode SDF system  
 $f_y$  = normalized yield strength  
 $\mu$  = ductility factor

$\omega_D$  = natural circular frequency of a damped vibration of a system

$\omega_n$  = the natural circular frequency of a system

## CHAPTER ONE- INTRODUCTION

Non linear static analysis, or pushover analysis, has been developed over the past twenty years and has become the preferred analysis procedure for design and seismic performance evaluation purposes as the procedure is relatively simple and considers post-elastic behaviour, [17].

However, the procedure involves certain approximations and simplifications that some amount of variation is always expected to exist in seismic demand predicting of pushover analysis, [17].

Although, in literatures, pushover analysis has been shown to capture essential structural response characteristics under seismic action, the accuracy and the reliability of pushover analysis in predicting global and local seismic demands for all structures have been a subject of discussion and improved procedures have been proposed to overcome the certain limitations of traditional pushover procedures. However, the improved procedures are mostly computationally demanding and conceptually complex that use of such procedures is impractical in engineering profession and codes, [23].

As traditional pushover analysis is widely used for design and seismic performance evaluation purposes, its limitation, weaknesses and the accuracy of its prediction in routine application should be identified by studying the factors affecting the pushover predictions. In other words, the applicability of pushover analysis in predicting seismic demands should be investigated for low, mid and high rise structures by identifying certain issues such as modelling nonlinear member behaviour, computational scheme of the procedure, variations in the predictions of various lateral load patterns in representing higher mode effects and accurate estimation of target displacement at which seismic demand prediction of pushover procedure is performed, [23].

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). Also, in force-controlled pushover procedure some numerical problems that affect the accuracy of results occur since target displacement may be associated with a very small positive or even a negative lateral stiffness because of the development of mechanisms and p-delta effects, [22].

Pushover analysis has been the preferred method for seismic performance evaluation of structures by the major rehabilitation guidelines and codes because it is conceptually simple. Pushover analysis allows tracing the sequence of yielding and failure on member and structural level as well as the progress of overall capacity curve of the structure, [23].

## **1.1 OBJECTIVE**

### **1.1.1 GENERAL OBJECTIVE**

To evaluate the role of shear wall on the performance of frame structure and also to see the effect of changing the value of gravitational acceleration from 0.05g (old Ethiopian building code standard) to 0.1g (new Ethiopian building code standard) on the performance of existing structures. And to find possible solution for existing reinforced frame structure.

### **1.1.2 SPECIFIC OBJECTIVE**

- To carry analysis to the effect of shear wall on the performance of reinforced concrete frame.
- By taking a case study building located in Addis Ababa city and assess its performance for the revised peak ground acceleration of EBCS-8 and propose a feasible engineering solution for the same structures with a similar complexity.
- To verify the reliability of the pushover analysis approach of softwares ( ETABS and SAP) in structural performance evaluation using conventional pushover analysis.

## **1.2 SCOPE OF THE STUDY**

This study is considers only for mid rise building, specifically forty –sixty common use residential building. For assessment of the case study working drawing is used. Any type of damage is not considered and so the following parameters are neglected on the analysis

- ❖ Existing damages
- ❖ Soil structure interaction
- ❖ Bond slips

Soil type is considered for the plot of the capacity curve and also when the capacity is compared with the demand. Any premature failure like shear failure is not expected to happen and if a member is shear liable, then it will be checked at that specific member.

### 1.3 STATEMENT OF THE PROBLEM

Many researchers have strived on performance determination of mid rise frame structure using pushover analysis , but the effect of shear wall on the performance of the frame is not clearly demarcated. So, does the present of shear wall significantly change the performance of frame structure? And also what will be the effect of changing the value of gravitational acceleration from 0.05g (old Ethiopian building code standard) to 0.1g (new Ethiopian building code standard) to the performance of the existing structure?

### 1.4 THESIS ORGANIZATION

The thesis is organized as per the briefing given below

**Chapter 1:** Gives general introduction, need for the investigating, objective and scope of the investigation and briefing of the organization of the thesis.

**Chapter 2:** Discusses the literature review i.e. the work done by various researchers in the field of modelling of structural members by pushover analysis.

**Chapter 3:** In this chapter, the philosophy of pushover analysis is presented, together with the demonstration of linear and non linear static methods of structural analysis has been discussed in detail. In addition, various methods used for determination of target displacement and performance point are also illustrated.

**Chapter 4:** present non linear analysis analytical modelling of structural components subjected to earthquake motions. Geometric and material non linearity of structural elements is explained. The numerical modelling of the frame building using computers soft was explained. Non linear capabilities using the plastic hinge concept and the tools needed for pushover analysis are also described.

**Chapter 5:** The general sequence of steps needed to perform pushover is explained. The results of the analysis are compiled and presented in appropriate manner. Comparison between the result obtained from Linear, Nonlinear building with and without shear wall plus changing the value of gravitational acceleration from old to new Ethiopian building code standard have been carried out. In addition the corresponding results obtained by response spectrum analysis are also presented. The seismic demands: floor displacements profiles, storey shear forces and inter-storey drifts ratios for each RC building are presented and discussed.

**Chapter 6:** Finally, salient conclusions and recommendations of the present study are given in this chapter followed by the references.

## CHAPTER TWO – LITERATURE REVIEW

### 2.1 GENERAL

To provide a detailed review of the literature related to modelling of structures in its entirety would be difficult to address in this chapter. A brief review of previous studies on the application of the pushover analysis of structures is presented in this section.

This literature review focuses on recent contributions related to pushover analysis of structures and past efforts most closely related to the needs of the present work.

### 2.2 LITERATURE REVIEW ON PUSHOVER ANALYSIS

**Berhanemeskel [5]** tried to investigate in his paper about performance assessment of reinforced concrete planar frame using nonlinear analysis by taking low rise buildings, specifically condominium building in Addis Ababa as case study and reached on the conclusion as follow.

- The increased value of the peak ground acceleration on the revised building code (EBCS-EN-2014) has negative effect on the structural performance of existing condominium buildings, therefore they should be investigated thoroughly and appropriate mitigation must be done in order to avoid catastrophic failure during occurrence of the expected seismic excitation.
- Static nonlinear pushover analysis is found to be reliable in predicting the performance of constructed precast condominium housings.
- Pushover analysis is advantageous not only in predicting the real time response of the structures but also in indicating progressive hinge formation. The progressive hinge formation can be utilized as a valuable input for strengthening purpose.
- As the study indicates, performance assessment of an existing structure should not rely on code provisions as code are very conservative which will undermine the actual capacity and fails to give the real time response of structures.
- The performance of a given structure depends on plastic hinge definition, the use of plastic hinge zone results in a better performance of the structure as compared with hinge points.

**Abdi [2]** in his research tried to study different analysis method such as conventional pushover analysis, modal pushover analysis and response spectrum analysis, of low and mid-rise concrete moment frame structures on stiff soil with existed plan irregularity.

The results obtained show that:

- With sufficient number of ‘modal’ pairs included, the height-wise distribution of storey shear, storey displacement and inter-storey drifts estimated by MPA is superior to the conventional pushover analysis (first ‘modal’ pair) result.
- Conventional pushover analysis yields realistic results for five storey regular, fair results in ten storey regular and poor results in ten storey irregular RC buildings. Therefore, the accuracy of the CPA decreases as the height and irregularities of the building increases.
- The contribution of the first two ‘modal’ pair suffices in most of the cases in MPA and is enough for intended purpose.
- The selected RC buildings met the limits states of design and serviceability (drift) earthquake requirements.
- If equal number of modes is used in RSA and MPA methods, MPA can give higher results for both regular and irregular buildings pushed in to the inelastic range.
- The CPA procedure found to provide close estimates of storey displacement profiles to that estimated by the MPA procedure.
- The contribution of higher modes to the storey shear is significant in both regular and irregular ten storey buildings which cannot be captured by CPA.
- P- $\Delta$  effects due to gravity loads in CPA and MPA procedures are small since the buildings are not deformed far into the inelastic range, therefore significant degradation in lateral capacity of the structures is not caused due to gravity load.

**Daniel [10]** explained the seismic vulnerability of selected buildings constructed in Addis Ababa was assessed based on three different bedrock acceleration ratios. Non linear dynamic analysis was also performed on four different reinforced concrete structure using IDARC 2D software. And in his paper he has given the conclusion that can be offered as recommendation for further research:

- It is recommended that nonlinear dynamic analysis procedures shall be used for seismic evaluation of existing buildings.
- Out of the four case study buildings considered in this study, 50% are found to be insufficient to resist the earthquake load induced on them. Therefore the author strongly suggests the evaluation of existing buildings constructed in Addis Ababa because if an earthquake of the expected intensity occurs, the result may be catastrophic.
- It is observed that the input process of IDARC 2D needs too many lines in text format. And also gathering the output is time taking. Therefore, further developments need to be done on the pre-processor and post-processor to save time and avoid input errors.

- It is also recommended that the Ethiopian building code standards include provisions that address strengthening of buildings.
- Finally the author suggests that such a study shall further be conducted on large number of buildings located in different sub cities of Addis Ababa, in order to get a detailed and realistic damage map of the city.

**Dhileep [11]** explained the practical difficulties associated with the non linear direct numerical integration of the equations of motion leads to the use of non linear static pushover analysis of structures. Pushover analysis is getting popular due to its simplicity. High frequency modes and non linear effects may play an important role in stiff and irregular structures. The contribution of higher modes in pushover analysis is not fully developed. The behaviour of high frequency model responses in non linear seismic analysis of structures is not known. In his paper an attempt is made to study the behaviour of high frequency model responses in non linear seismic analysis of structures.

Non linear static pushover analysis used as an approximation to non linear time history analysis is becoming a standard tool among the engineers, researches and professionals worldwide. High frequency modes may contribute significantly in the seismic analysis of irregular and stiff structures. In order to take the contribution of higher modes structural engineers may include high frequency modes in the non linear static pushover analysis. The behaviour of high frequency modes in non linear static pushover analysis of irregular structures is studied. At high frequencies, the responses of non linear dynamic analysis converge to the non linear static pushover analysis. Therefore non linear response of high frequency modes can be evaluated using a non linear static push over analysis with an Implemental force pattern given by their modal mass contribution times zero period acceleration. The higher modes with rigid content as a major contributing factor exhibit a better accuracy in non linear pushover analysis of structures when compared to the damped periodic modes.

**Oscar [20]** explained the following conclusions that can be offered as suggestions for further research:

- Performance-based design in earthquake engineering implies consideration of the uncertainties in the structure.
- Demands and capacities, in order to evaluate the reliability associated with each of the required performance levels. These reliabilities must satisfy minimum target values for each level.

- Calculation of the structural responses for the formulation of the limit states equations requires a nonlinear dynamic analysis, and these responses cannot be given in an explicit relationship in terms of the intervening random variables. Discrete data can be obtained for chosen combinations of these variables, and the results can be expressed in terms of response surfaces or neural networks. In this work the latter approach has been followed, providing flexibility and adaptability.
- The major computational demand in this approach is the construction of the discrete database, executing the nonlinear dynamic analysis for a number of variable combinations representatives of the variable ranges. For a fixed combination within a sub-set of the variables, the analysis is carried out for another sub-set which groups variables including different ground motions. For each combination, and over the set of grouped variables, the mean and the standard deviation of each response of interest are obtained. These statistics are then represented by neural networks, and are utilized in representing the responses in a probabilistic manner.
- The utilization of neural networks' representation for the response demands makes feasible the calculation of the probability of non-performance via standard Monte Carlo simulation.
- The reliability associated with each performance level can thus be estimated for different combinations of design parameters, and these reliabilities can themselves be represented by neural networks.
- The optimization in performance-based design implies the minimization of an objective function (here the total structural cost was used) subject to the achievement of minimum target reliabilities at each performance level. This work has shown the implementation of an optimization scheme based on a search without calculation of gradients. This scheme is efficient, whether the intermediate reliability constraints are evaluated by simulation at each step, or they are implemented using the reliability neural networks.
- The optimization scheme for minimum total cost has been applied to a multi-storey, multi-bay reinforced concrete frame, with the design parameters being the depths of beams and columns, and three steel reinforcement ratios. The results show good agreement between the two ways of implementing the calculation of the reliability constraints, and that somewhat different optimum design parameters may correspond to minor differences in the total cost. In particular, the results have shown that it is important the consideration of damage repair costs, as they influence the optimum solution.

- This work has shown that neural networks offer a very useful tool to represent the relationship between structural responses and the intervening random variables, and between achieved reliabilities and the design parameters. The first application make feasible the use of Monte Carlo simulation to estimate reliabilities or probabilities of non-performance, while the second improves the efficiency of the optimization algorithm when intermediate reliabilities need to be evaluated.
- The approach presented introduced a general scheme for reliability estimation and performance-based design optimization in earthquake engineering. It introduced required concepts like a relationship between damage level and repair cost – a relationship that still needs further general development and should be the objective of continuing research.
- Continuing research should also be focused on damage parameters and their relationship to calculated quantities like strains and displacements. Here a well known damage index was used for the purpose of the application, but further research should be focused on how damage accumulates over time as a result of the applied strains or displacement history..

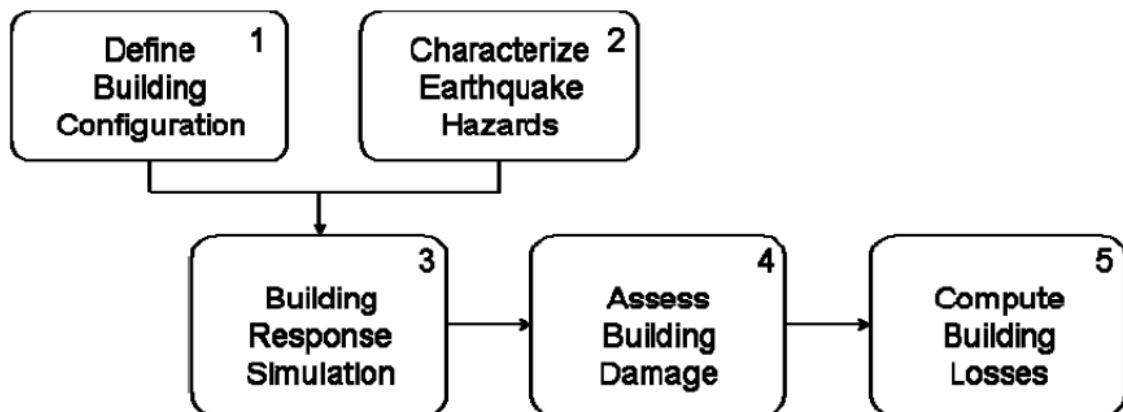
**A. Shuraim** [1] summarized the nonlinear static analytical procedure (Pushover) as introduced by ATC-40 has been utilized for the evaluation of existing design of a new reinforced concrete frame, in order to examine its applicability. Potential structural deficiencies in RC frame, when subjected to a moderate seismic loading, were estimated by the code seismic-resistant design and pushover approaches. In the first method the design was evaluated by redesigning under one selected seismic combination in order to show which members would require additional reinforcement. It was shown that most columns required significant additional reinforcement, indicating their vulnerability if subjected to seismic forces. On the other hand, the nonlinear pushover procedure shows that the frame is capable of withstanding the presumed seismic force with some significant yielding at all beams and one column. Vulnerability locations from the two procedures are significantly different. The paper has discussed the reasons behind the apparent discrepancy which is mainly due to the default assumptions of the method as implemented by the software versus the code assumptions regarding reduction factors and maximum permissible limits. In new building design, the code always maintains certain factor of safety that comes from load factors, materials reduction factors, and ignoring some post yielding characteristics (hardening). In the modelling assumptions of ATC-40, reduction factor is assumed to be one, and hardening is to be taken into consideration. Hence, the paper suggests that engineering judgment should be exercised prudently when using the pushover analysis and that engineer should follow the

code limits when designing new buildings and impose certain reductions and limits in case of existing buildings depending on their conditions. In short software should not substitute for code provisions and engineering judgment.

**A. Whittaker [24]** summarizes the next (second) generation tools and procedures for performance-based earthquake engineering in the United States. The methodology, which is described in detail in the draft Guidelines for the Seismic Performance Assessment of Buildings, builds on the first generation deterministic procedures, which were developed in the ATC-33 project in the mid 1990s and in ASCE Standard: ASCE/SEI 41-06 Seismic Rehabilitation of Existing Buildings.

The procedures and methodologies described in these guidelines include an explicit treatment of the large uncertainties in the prediction of losses due to earthquakes. This formal treatment of uncertainty and randomness represents a substantial advance in performance based engineering and a significant departure from the first generation deterministic procedures.

Fig.2.1 identifies the five basic steps proposed for a next-generation seismic performance assessment. Unlike prior assessment procedures that addressed either structural damage or repair cost, three measures of seismic performance are proposed in the guidelines: 1) direct economic loss (repair cost), 2) indirect economic loss (downtime or business interruption), and 3) casualties (including injuries and death). Each of three performance measures is treated as a potential loss. Section 2 of the paper introduces the three types of performance assessment that can be performed using the draft Guidelines and identifies the basic procedure for each. Section 3 describes the five steps for seismic performance assessment that are identified in Fig 2.1



**Figure 2.1 procedures for performance Assessment [24]**

The procedures set forth in these guidelines represent a substantial departure from the deterministic tools and procedures used at this time because uncertainty and randomness is captured explicitly in every step of the proposed procedures. Fragility functions, damage states and building-level consequence functions, are used in the proposed procedures to compute losses.

**Ceroni [6]** formulated that ductility of R.C. elements has been widely studied either experimentally and theoretical since its evaluation is basic to carry out a reliable non-linear analysis of structures; post-elastic deformability is a resource for redistributing stresses in a structure to increase the ultimate load but, above all, to absorb and dissipate energy during major earthquakes. However, the problem remains open and models still need an improvement in two directions. On one side, mechanical models can be implemented to take into account constructive details, shear-flexure interaction, size effects as well as non-linear constitutive relationship of materials and steel-concrete bond. On the other side, simplified approaches have to be assessed in order to allow an easy but reliable ductility evaluation without using any sophisticated analytical model, generally not very designers friendly. In this paper a wide parametric analysis with a refined model is carried out in order to build on a reliable formulation for the plastic hinge length of R.C. columns subjected to axial and flexural load. The model used to analyse the non-linear behaviour of the element and to estimate the plastic rotation is a point by point model, including an explicit formulation of the bond slip relationship and capable to take into account the effect of the distributed and concentrated non-linearity, as the spread of plasticity along the member and the fixed end rotation. Its efficiency has been already successfully applied to experimental comparison (Cosenza, E. et al., 1998).

The rotational capacity evaluated by the model varying some parameters allows a clear understanding of the futures influence involved in the structural problem. Ductility of R.C elements depends on behaviour of the cracked section, which is well represented by moment curvature relationship; the ratio of ultimate curvature to the one at first yielding is called section ductility. If the rotational capacity has to be calculated in actual cases, models based on the evaluation of a plastic hinge length are very useful thanks to their procedure simplicity. It is therefore surely interesting to review the evaluation of the plastic hinge length  $L_p$  using the detailed model.

$$L_p = L_{pI} + L_{pII} \quad (2.1)$$

Where  $L_{pI}$  is due to the plastic rotation of the column and  $L_{pII}$  to the fixed end rotation at the footing zone of the column. In order to extrapolate a formulation for  $L_{pI}$  and  $L_{pII}$ , a wide

parametric analysis has been developed in the same hypothesis explained in the previous paragraph. The column considered has length  $L$  equal to 1.5 m, 2 m, 2.5 m, 3 m and a square cross section with side  $H$  equal to 30 and 60 cm symmetrically reinforced; the combination of values of  $L$  and  $H$  gives back, for the ratio  $L/H$ , the values of 3.33, 5, 6.67, 8.33 and 10.

The concrete strength in compression is  $f_c=30$  MPa and the volumetric percentage of stirrups is 0.1%. The ratio  $f_t/f_y$  does vary in the range 1.05-1.45; the ultimate strain of steel  $\epsilon_u$  does vary in the range 0.04-0.16. Three diameters of steel bar,  $d_b$ , (10, 16 and 20 mm) are considered. The values of the ratio  $N/N_u$  considered are 0, 0.1, 0.2, 0.3, and 0.4.

$$L_P^I = 6.1 \cdot (L/H)^{0.43} \cdot (f_t/f_y - 1)^{0.65} \cdot \epsilon^{-0.32} \cdot (1+N/N_0)^{-1.83} \quad (2.2)$$

$$L_P^{II} = 5 \cdot d_b \cdot (f_t/f_y - 1)^{0.2} \quad (2.3)$$

The influence on the plastic hinge of the ratio between an element typical length (distance of critical section to the point of contra flexure, shear span...) and the section height has been already pointed out by (Baker. et., al, 1965), who also explicitly introduced the influence of the ratio  $N/N_u$ , while the steel properties and concrete strength were considered as factors for mild and cold-worked steel. Since then, laying on experimental results and empirical considerations, other expressions have been proposed aimed to simplify the formulation of  $L_p$  reducing the number of parameters and considering only the influence of geometrical properties of an element (length, height of section). The influence of steel bar diameter was taken into account by (Priestley and Park. 1987), based on the analysis of experimental tests on 20 columns:

$$L_p = 0.08L + 6d_b \quad (2.4)$$

Where  $L$  is the distance from the point of contra flexure of the column to the section of maximum moment and  $d_b$  the bars diameter; the first and second terms of the formulation represent the  $L_{pI}$  and  $L_{pII}$  contributions, both independent from the steel characteristics. The variables examined in the experimental tests were the section shape (square, rectangular and circular cross section), the longitudinal and lateral reinforcement content and the loading rate. The effect of axial load and steel properties was not analysed.

Later on, in (B.I.A. 1996) a modification of the previous expression was proposed introducing the effect of the steel yielding stress:

$$L_p = 0.08L + 0.022f_y d_b \quad (2.5)$$

Recently in (Fib Bulletin. 2003) formulations similar to last eqn. for monotonic and cyclic loads have been suggested, as follows:

$$\text{For monotonic loads: } L_p = 0.18L_s + 0.025f_y d_b \quad (2.6)$$

$$\text{For cyclic loads: } L_p = 0.08L_s + 0.017f_y d_b \quad (2.7)$$

Where  $L_s$  is the shear span

At last he concluded in his formulation that the availability of a reliable formulation for the plastic hinge length is a key issue for any analysis of R.C. element ductility, i.e. to non-linear behaviour of R.C. frames under seismic actions. The proposed formulation is based on a wide numerical analysis developed through a detailed mechanical model which takes into account the non-linear constitutive relationship of material and the steel-concrete bond law. It allows considering the effect of yielding penetration between cracks of the structures but also at the steel anchorage in the foundation.

In particular, the two contributions to the plastic deformability of a column can be separately evaluated multiplying the respective plastic length by the curvature of the section at the element base; thus the element ductility can be easily evaluated knowing the section behaviour. The formulation in terms of plastic rotation takes into account many parameters and shows a low scatter respect to the numerical results; furthermore the influence of parameters appears in agreement with the mechanical behaviour. The range of some parameters considered to assess the proposal is wider than the ones used in experimental tests at the base of other available formulations, but it is limited to cold formed steel and elements without shear-flexure interaction; therefore the analysis has to be extended developing an experimental comparison too.

**Chung-Yue [8]** in his paper he presented a method for the determination of the parameters of plastic hinge properties (PHP) for structure containing RC wall in the pushover analysis is proposed. Nonlinear relationship between the lateral shear force and lateral deformation of RC wall is calculated first by the Response-2000 and Membrane-2000 code. The PHP (plastic hinge properties) value of each parameter for the pushover analysis function of SAP2000 or ETABS is defined as the product of two parameters  $\alpha$  and  $\beta$ . Values of  $\alpha$  at states of cracking, ultimate strength and failure of the concrete wall under shear loading can be determined

respectively from the calculations by Response-2000. While the corresponding  $\beta$  value of each PHP parameter is obtained from the regression equations calibrated from the experimental results of pushover tests of RC frame-wall specimens. The accuracy of this newly proposed method is verified by other experimental results. It shows that the presented method can effectively assist engineers to conduct the performance design of structure containing RC shear wall using the SAP2000.

SAP2000 is a well known and widely accepted, general-purpose, three-dimensional structural analysis program. The pushover analysis module has been installed into the SAP2000. In the procedure of the pushover analysis, the assignment of the values of plastic hinge properties (PHP) strongly affects the prediction of the capacity curve of RC structure.

SAP2000 program includes several built-in default hinge properties that are based on average values from ATC-40 for concrete members. These built-in properties can be useful for preliminary analyses, but user-defined properties are recommended for final analyses (Habibullah and Pyle, 1998). Yielding and post-yielding behaviour can be modelled using discrete user-defined hinges. Currently SAP2000 allows hinges can only be introduced into frame elements; the PHP properties can be assigned to a frame element at any location along it. The authors have been developed a dual parameters method to define the PHP properties of RC frame structure for the pushover analysis (Ho and Wang, 2006). The purpose of this paper is to extend the application of this method to the RC structures containing RC shear wall. In order to use the functions provided by the SAP2000 code, the RC shear wall is treated as a wide, flat column. Modelling a RC wall as a wide and flat column (frame elements) not only can consider the steel reinforcements in RC elements exactly, but also can assign the PHP of RC walls according to its plastic behaviour. In SAP2000, the default properties are available for hinges in the following degrees of freedom:

1. Axial (P)
2. Major shear (V2)
3. Major moment (M3)
4. Coupled P-M2-M3 (PMM)

He concluded that a dual parameters method is introduced to define the plastic hinge properties (PHP) of RC wall in the pushover analysis of RC structure. The effectiveness of this simple method is verified by the agreement of the prediction curves with some additional test data. This newly proposed method is quite simple and is easy for engineers to link with commercial structural analysis code to conduct the performance design of structure under seismic loading.

**Konuralp [17]** explained that structural frames are often filled with in filled walls serving as partitions. Although the infills usually are not considered in the structural analysis and design, their influence on the seismic behaviour of the infilled frame structures is considerable. In this study, a parametric study of certain infilled frames, using the strut model to capture the global effects of the infills was carried out. Three concrete planar frames of five-stories and three-bays are considered which have been designed in accordance with Turkish Codes. Pushover analysis is adopted for the evaluation of the seismic response of the frames. Each frame is subjected to four different loading cases. The results of the cases are briefly presented and compared. The effect of infill walls on seismic behaviour of two sample frames with different infill arrangements was investigated. The results yield that it is essential to consider the effect of masonry infills for the seismic evaluation of moment-resisting RC frames, especially for the prediction of its ultimate state, infills having no irregularity in elevation have beneficial effect on buildings and infills appear to have a significant effect on the reduction of global lateral displacements.

Infills have been generally considered as non-structural elements, although there are codes such as the Eurocode-8 that include rather detailed procedures for designing infilled R/C frames, presence of infills has been ignored in most of the current seismic codes except their weight. However, even though they are considered non-structural elements the presence of infills in the reinforced concrete frames can substantially change the seismic response of buildings in certain cases producing undesirable effects (torsional effects, dangerous collapse mechanisms, soft storey, variations in the vibration period, etc.) or favourable effects of increasing the seismic resistance capacity of the building.

The pushover analysis can be considered as a series of incremental static analyses carried out to examine the non-linear behaviour of structure, including the deformation and damage pattern. The procedure consists of two parts. First, a target displacement for the structure is established. The target displacement is an estimate of the seismic top displacement of the building, when it is exposed to the design earthquake excitation. Then, a pushover analysis is carried out on the structure until the displacement at the top of the building reaches the target displacement. The extent of damage experienced by the building at the target displacement is considered to be representative of the damage experienced by the building when subjected to design level ground shaking. A judgment is formed as to the acceptability of the structural behaviour for the design of the new building, or the level of damage of an existing building for evaluation purposes.

In the conclusion he states that the effect of infill walls on seismic behaviour of a two sample frames with different infill arrangements was investigated. The results yield the following conclusions.

- It is essential to consider the effect of masonry infill's for the seismic evaluation of moment resisting RC frames, especially for the prediction of its ultimate state.
- Infills having no irregularity in elevation have beneficial effect on buildings. In infilled frames with irregularities, such as soft story, damage was found to concentrate in the levels where the discontinuity occurs.
- Since infills increases lateral resistance and initial stiffness of the frames they appear to have a significant effect on the reduction of the global lateral displacement.
- Arrangement of infills may affect the post yield behaviour and has an influence on distribution and sequence of damage formation. To generalize this, more infill arrangements should be investigated.
- A carefully performed pushover analysis can provide insight into structural aspects that control performance of the structure during a severe earthquake.
- The choice of the static load distribution used in pushover analysis can affect the accuracy of the response estimates..

**A. K. Chopra [7]** extracted an improved Direct Displacement-Based Design Procedure for Performance-Based seismic design of structures. Direct displacement-based design requires a simplified procedure to estimate the seismic deformation of an inelastic SDF system, representing the first (elastic) mode of vibration of the structure. This step is usually accomplished by analysis of an “equivalent” linear system using elastic design spectra. In their work, an equally simple procedure is developed that is based on the well-known concepts of inelastic design spectra. This procedure provides: (1) accurate values of displacement and ductility demands, and (2) a structural design that satisfies the design criteria for allowable plastic rotation. In contrast, the existing procedure using elastic design spectra for equivalent linear systems is shown to underestimate significantly the displacement and ductility demands.

In this work, it is demonstrated that the deformation and ductility factor that are estimated in designing the structure by this procedure are much smaller than the deformation and ductility demands determined by nonlinear analysis of the system using inelastic design spectra. Furthermore, it has been shown that the plastic rotation demand on structures designed by this procedure may exceed the acceptable value of the plastic rotation.

## CHAPTER THREE- PUSHOVER ANALYSES

### 3.1 GENERAL

Pushover Analysis option will allow engineers to perform pushover analysis as per FEMA - 356 and ATC-40. Pushover analysis is a static, nonlinear procedure using simplified nonlinear technique to estimate seismic structural deformations. It is an incremental static analysis used to determine the force-displacement relationship, or the capacity curve, for a structure or structural element. The analysis involves applying horizontal loads, in a prescribed pattern, to the structure incrementally, i.e. pushing the structure and plotting the total applied shear force and associated lateral displacement at each increment, until the structure or collapse condition, [23].

Pushover analysis is a technique by which a computer model of the building is subjected to a lateral load of a certain shape (i.e., inverted triangular or uniform). The intensity of the lateral load is slowly increased and the sequence of cracks, yielding, plastic hinge formation, and failure of various structural components is recorded. Pushover analysis can provide a significant insight into the weak links in seismic performance of a structure. A series of iterations are usually required during which, the structural deficiencies observed in one iteration, are rectified and followed by another. This iterative analysis and design process continues until the design satisfies pre-established performance criteria. The performance criteria for pushover analysis are generally established as the desired state of the building given roof-top or spectral displacement amplitude.

Static Nonlinear Analysis technique, also known as sequential yield analysis, or simply “pushover” analysis has gained significant popularity during the past few years. It is the one of the three analysis techniques recommended by FEMA-273/274 and a main component of the Spectrum Capacity Analysis method (ATC-40). Proper application can provide valuable insights into the expected performance of structural systems and components. Misuse can lead to an erroneous understanding of the performance characteristics. Unfortunately, many engineers are unaware of the details that have to observe in order to obtain useful results from such analysis, [27].

### 3.2 LIMITATIONS OF PUSHOVER ANALYSIS

Although pushover analysis has advantages over elastic analysis procedures, underlying assumptions, the accuracy of pushover predictions and limitations of current pushover procedures must be identified. The estimate of target displacement, selection of lateral load patterns and identification of failure mechanisms due to higher modes of vibration are important issues that affect the accuracy of pushover results, [25].

Target displacement is the global displacement expected in a design earthquake. The roof displacement at mass centre of the structure is used as target displacement. The accurate estimation of target displacement associated with specific performance objective affect the accuracy of seismic demand predictions of pushover analysis.

However, in pushover analysis, generally an invariant lateral load pattern is used that the distribution of inertia forces is assumed to be constant during earthquake and the deformed configuration of structure under the action of invariant lateral load pattern is expected to be similar to that experienced in design earthquake. As the response of structure, thus the capacity curve is very sensitive to the choice of lateral load distribution, selection of lateral load pattern is more critical than the accurate estimation of target displacement,[25].

The lateral load patterns used in pushover analysis are proportional to product of story mass and displacement associated with a shape vector at the story under consideration. Commonly used lateral force patterns are uniform, elastic first mode, "code" distributions and a single concentrated horizontal force at the top of structure. Multi-modal load pattern derived from Square Root of Sum of Squares (SRSS) story shears is also used to consider at least elastic higher mode effects for long period structures. These loading patterns usually favour certain deformation modes that are triggered by the load pattern and miss others that are initiated and propagated by the ground motion and inelastic dynamic response characteristics of the structure. Moreover, invariant lateral load patterns could not predict potential failure modes due to middle or upper story mechanisms caused by higher mode effects. Invariant load patterns can provide adequate predictions if the structural response is not severely affected by higher modes and the structure has only a single load yielding mechanism that can be captured by an invariant load pattern,[25].

FEMA-273 recommends utilising at least two fixed load patterns that form upper and lower bounds for inertia force distributions to predict likely variations on overall structural behaviour and local demands. The first pattern should be uniform load distribution and the other should be "code" profile or multi-modal load pattern. The 'Code' lateral load pattern is

allowed if more than 75% of the total mass participates in the fundamental load. The invariant load patterns cannot account for the redistribution of inertia forces due to progressive yielding and resulting changes in dynamic properties of the structure. Also, fixed load patterns have limited capability to predict higher mode effects in post-elastic range. These limitations have led many researchers to propose adaptive load patterns which consider the changes in inertia forces with the level of inelasticity. The underlying approach of this technique is to redistribute the lateral load shape with the extent of inelastic deformations. Although some improved predictions have been obtained from adaptive load patterns, they make pushover analysis computationally demanding and conceptually complicated. The scale of improvement has been a subject of discussion that simple invariant load patterns are widely preferred at the expense of accuracy. Whether lateral loading is invariant or adaptive, it is applied to the structure statically that a static loading cannot represent inelastic dynamic response with a large degree of accuracy, [25].

### 3.3 VARIOUS HINGE MODELS OF PUSHOVER ANALYSIS

These are the various hinge models used in pushover analysis:

According to the Ceroni et., al, (2007) the rotational capacity of the element can be defined as the plastic fraction  $\theta_p$  of the rotation  $\theta_u$  at failure. It can be evaluated as the difference between the rotation at the maximum moment and the rotation at the steel yielding  $\theta_y$ :

$$\theta_p = \theta_u - \theta_y \quad (3.1)$$

The plastic rotation must include the contribution of the fixed end rotation  $\theta_{p,fix}$ ,

$$\theta_p = \theta_{p,c} - \theta_{p,fix} \quad (3.2)$$

The fixed end rotation  $\theta_{p,fix}$ , is evaluated as the ratio between the slip of the tensile bars at the column base and the neutral axis depth of the base section. The value of  $\theta_{p,fix}$  depends on all the parameters introduced, but above all the steel characteristics and the bond-slip relation are important; moreover the bar diameter has to be considered, for its influence on bond. The term  $\theta_{p,c}$  represents the contribute to plastic rotation of column deformability.

If the rotational capacity has to be calculated in actual cases, models based on the evaluation of a plastic hinge length are very useful thanks to their procedure simplicity. It is therefore

Surely interesting to review the evaluation of the plastic hinge length  $L_p$  using the detailed model previously introduced.

The plastic hinge length can be obtained dividing the plastic rotation  $\theta_p$  to the plastic curvature  $\phi_p$ :

$$L_p = \theta_p / \phi_p \quad (3.3)$$

$$\phi_p = \phi_U - \phi_Y \quad (3.4)$$

$$\theta_p = \theta_U - \theta_Y = (\phi_U - \phi_Y) \cdot L_p \quad (3.5)$$

Due to the fixed end rotation, the  $L_p$  value can be divided into two contributions:

$$L_p = L_{pI} + L_{pII} \quad (3.6)$$

Where  $L_{pI}$  is due to the plastic rotation of the column and  $L_{pII}$  to the fixed end rotation at the footing zone of the column.

The following expressions for  $L_{pI}$  and  $L_{pII}$  have been obtained:

$$L_{pI} = 6.1 \cdot (L/H)^{0.43} \cdot (f_t/f_y - 1)^{0.65} \cdot \epsilon^{-0.32} \cdot (1+N/N_0)^{-1.83} \quad (3.7)$$

$$L_{pII} = 5 \cdot d_b \cdot (f_t/f_y - 1)^{0.2} \quad (3.8)$$

According to **Priestley et al, (1987)** the plastic hinge length formula is:

$$L_p = 0.08L + 6d_b \quad (3.9)$$

Where  $L$  is the distance from the point of contra flexure of the column to the section of maximum moment and  $d_b$  the bars diameter;

According to **B.I.A. 1996**, the plastic hinge length formula is:

$$L_p = 0.08L + 0.022 f_y d_b \quad (3.10)$$

According to **Bulletin of TG7.2, (2003)** the formula of plastic hinge length:

$$\text{For monotonic loads: } L_p = 0.18 \cdot L_s + 0.025 \cdot f_y \cdot d_b \quad (3.11)$$

$$\text{For cyclic loads: } L_p = 0.08 \cdot L_s + 0.017 \cdot f_y \cdot d_b \quad (3.12)$$

Where  $L_s$  is the shear span.

According to **Bulletin of TG7.2, (2003)** the ultimate rotation  $\theta_u$  calculated according to the following equation:

$$\theta_u = \theta_y + (\theta_U - \theta_U). L_p \cdot \{1 - 0.5 \cdot L_p / L_s\} \quad (3.13)$$

The ultimate and yielding curvatures were calculated using the section equilibrium equations and considering a constitutive relationship for the confined concrete. Rotation at steel yielding,  $\theta_y$ , was calculated through an empirical expression statistically fitted to the experimental results on beams, columns and walls.

According to Priestley et. al, (1996) the ultimate concrete compressive strain can be calculated by:

$$\epsilon_{cu} = 0.004 + 1.4 \rho_s f_{yh} \epsilon_{cu} / f_{cc} \quad (3.14)$$

where  $\epsilon_{cu}$  is the ultimate concrete compressive strain,  $\epsilon_{su}$  is the steel strain at the maximum tensile stress,  $\rho_s$  is the volumetric ratio of confining steel,  $f_{yh}$  is the yield strength of transverse reinforcement, and  $f_{cc}$  is the peak confined concrete compressive strength.

### 3.4 ELEMENT DESCRIPTION OF SAP2000/ETAB2015

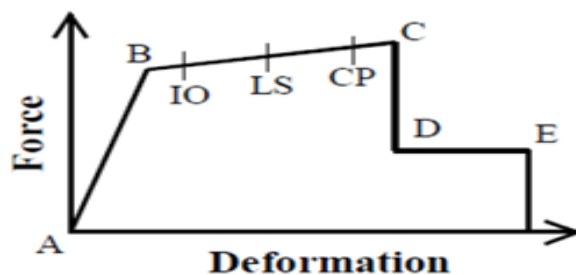


Fig.3.1 Force-Deformation for Pushover Hinge [15]

Point A corresponds to unloaded condition and point B represents yielding of the element. The ordinate at C corresponds to nominal strength and abscissa at C corresponds to the deformation at which significant strength degradation begins. The drop from C to D represents the initial failure of the element and resistance to lateral loads beyond point C is usually unreliable. The residual resistance from D to E allows the frame elements to sustain gravity loads. Beyond point E, the maximum deformation capacity, gravity load can no longer be sustained. Hinges can be assigned at any number of locations (potential yielding points) along the span of the frame element as well as element ends. Uncoupled moment ( $M_2$  and  $M_3$ ), torsion (T), axial force (P) and shear ( $V_2$  and  $V_3$ ) force-displacement relations can be defined. As the column axial load changes under lateral loading, there is also a coupled P- $M_2$ - $M_3$  (PMM) hinge which yields based on the interaction of axial force and bending moments at the hinge location. Also, more than one type of hinge can be assigned at the same location of a frame element. There are three types of hinge properties in ETABS 2015. They are default hinge properties, user-defined hinge properties and generated hinge properties. Only default hinge properties and user-defined hinge properties can be assigned to frame elements, [15].

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

Default hinge properties could not be modified and they are section dependent. When default hinge properties are used, the program combines its built-in default criteria with the defined section properties for each element to generate the final hinge properties. The built-in default hinge properties for steel and concrete members are based on ATC-40 and FEMA-273 criteria.

User-defined hinge properties can be based on default properties or they can be fully user-defined. When user-defined properties are not based on default properties, then the properties can be viewed and modified. The generated hinge properties are used in the analysis. They could be viewed, but they could not be modified, [15].

### **3.5 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, [23].

#### **3.5.1 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 smoothed 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 effect 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, [23].

### **3.5.2 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 Modal Pushover Analysis (MPA) method, [23].

### **3.5.2.1 Method of inelastic static analysis to evaluate seismic performance**

Many methods were presented to apply the inelastic static procedure to structures. The commonly used methods can be listed as

- (1) The Capacity Spectrum Method(CSM) (ATC,1996)
- (2) The Displacement Coefficient Method (DCM) (FEMA-273/356,1997)
- (3) Modal Pushover Analysis (MPA) method

#### **3.5.2.1.1 The Capacity Spectrum Method (CSM)**

Capacity Spectrum Method is one of the most popular methods utilized for a quick estimate to evaluate the seismic performance of structures. The method is recommended by ATC-40 as a displacement-based design and assessment tool for structures. The method was developed by freeman and it has gone through several modifications since then. The most recent three versions (Procedures A, B and C) of Capacity Spectrum Method are presented in detail in ATC-40. The method requires construction of a structure capacity curve and its comparison with the estimated demand response spectrum, both of which are expressed in Acceleration-Displacement Response Spectrum (ADRS) format.

#### **Design Response Spectra**

The selection of a performance objective involves the specification of a hazard level. Unless ground motion time histories are used in a dynamic time-history analysis, it is customary to specific the hazard in terms of a response spectrum. The generation of the ground motion hazard spectrum is a function of several parameters, most of which pertain to site characteristics.

In the absence of ground motion time histories, both ATC-40 and FEMA-356 have presented a procedure to construct the elastic design spectra.

#### **Generating the Design Spectrum**

Elastic design site response spectra are described by a standard (two domain) shape defined by the coefficients  $C_A$  and  $C_V$ . Elastic response spectra are described by a standard shape to simplify the application of these spectra to nonlinear static analysis procedures. The procedure to construct the elastic design spectrum can be summarized as shown in figure 3.2

#### **ATC-40 Provisions for Generating the Design Spectrum**

$C_A$ -a site response coefficient, simply the effective peak acceleration (EPA) at the site

$C_V$ - a coefficient when divided by the period defines the acceleration in the constant velocity domain.

ATC-40 provides the three options when developing the elastic design spectra. But SAP2000 uses the site seismic coefficients given in table 3.2

To use Table 3.2, it is first necessary to determine the shaking intensity, which is defined as the product of three quantities:  $Z * E * N$  = zone factor\* earthquake hazard level\*near source factor.

Using the shaking intensity value, the corresponding site response coefficient  $C_A$  and  $C_V$  are obtained for a known soil profile. The response spectrum can be easily generated as indicated in figure 3.2.

TABLE 3.1 Average Soil Properties Used to Establish Soil Profile [3]

Table 4-3. Soil Profile Types

Soil Profile Type	Soil Profile Name/Generic Description	Average Soil Properties for Top 100 Feet of Soil Profile		
		Shear Wave Velocity, $\bar{v}_s$ (feet/second)	Standard Penetration Test, $\bar{N}$ (or $N_{60}$ ) for cohesionless soil layers (blows/foot)	Undrained Shear Strength, $\bar{s}_u$ (psf)
Sa <sup>1</sup>	Hard Rock	$\bar{v}_s > 5,000$	Not Applicable	
Sb	Rock	$2,500 < \bar{v}_s \leq 5,000$	Not Applicable	
Sc	Very Dense Soil and Soft Rock	$1,200 < \bar{v}_s \leq 2,500$	$\bar{N} > 50$	$\bar{s}_u > 2,000$
Sd	Stiff Soil Profile	$600 \leq \bar{v}_s \leq 1,200$	$15 \leq \bar{N} \leq 50$	$1,000 \leq \bar{s}_u \leq 2,000$
Se <sup>2</sup>	Soft Soil Profile	$\bar{v}_s < 600$	$\bar{N} < 15$	$\bar{s}_u < 1,000$
Sf <sup>3</sup>	Soil Requiring Site-Specific Evaluation			

- 1 Soil profile type Sa (hard rock) is not applicable to sites in California.
- 2 Soil profile type Sc also includes any soil profile with more than 10 feet of soft clay defined as a soil with  $PI > 20$ ,  $w_{mc} \geq 40\%$  and  $\bar{s}_u < 500$  psf. The plasticity index (PI) is determined in accordance with ASTM D4318-93 and the moisture content ( $w_{mc}$ ) is determined in accordance with ASTM D2216-92.
- 3 See Section 4.4.1.2 for description of soils requiring site-specific evaluation.

Table 3.2. ATC-specifications for developing hazard spectrum [3]

Table 4-8. Seismic Coefficient,  $C_v$

Soil Profile Type	Shaking Intensity, $ZEN^{0.2}$					
	$\approx 0.075$	$\approx 0.15$	$\approx 0.20$	$\approx 0.30$	$\approx 0.40$	$> 0.40$
Sb	0.08	0.15	0.20	0.30	0.40	1.0(ZEN)
Sc	0.13	0.25	0.32	0.45	0.56	1.4(ZEN)
Sd	0.18	0.32	0.40	0.54	0.64	1.6(ZEN)
Se	0.26	0.50	0.64	0.84	0.96	2.4(ZEN)
Sf	Site-specific geotechnical investigation required to determine $C_v$					

- 1 The value of E used to determine the product, ZEN, should be taken to be equal to 0.5 for the Serviceability Earthquake, 1.0 for the Design Earthquake and 1.25 (Zone 4 sites) or 1.5 (Zone 3 sites) for the Maximum Earthquake.
- 2 Seismic coefficient  $C_v$  should be based on the linear interpolation of values for shaking intensities other than those shown in the table.

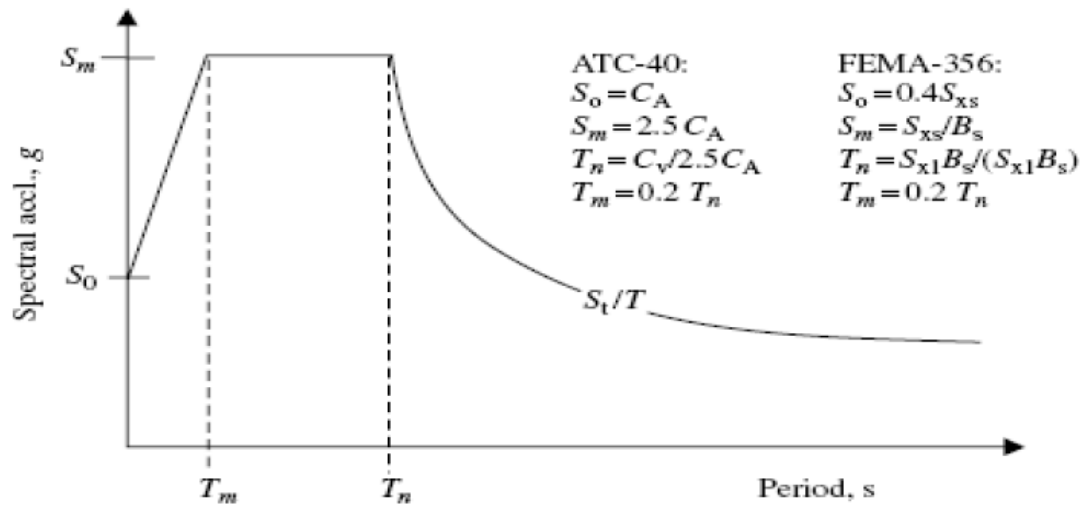


Figure 3.2 ATC-40 and FEMA-356 representation of the 5% damped response spectrum  
 If the elastic design spectrum is used to create the demand spectrum, the overlay is valid only if the structural response is also elastic. Hence, the next step in the process is to reduce the elastic response spectrum to an inelastic spectrum using the concept of equivalent damping. Using the fundamental principles of the structural mechanics, the equivalent damping  $\zeta$  associated with dissipated energy during inelastic response is given by

$$\zeta_{eq} = 1/(\Omega/\omega)(1/4\pi)(E_D/E_S) \tag{3.15}$$

where  $(\Omega/\omega)$  is the ratio of the forcing frequency to the natural frequency of the system,  $E_D$  is the energy dissipated through hysteretic behaviour, and  $E_S$  is the strain energy at the maximum displacement. If it is assumed that the peak response is associated with the resonant frequency, then the ratio  $(\Omega/\omega) = 1$

The ATC-40 methodology for estimating the equivalent viscous damping is derived for a bilinear capacity curve, therefore, it is necessary to transfer the capacity curve into bilinear form.

The elastic design spectrum already incorporates 5% damping; hence the equivalent damping from inelastic behaviour must be added to the elastic viscous damping. For behaviour other than bilinear hysteresis, a modification factor  $\kappa$  is introduced.

The final damping value incorporating elastic damping, equivalent inelastic damping, and general hysteretic behaviour is given by

$$Z_{\text{eff}} = \kappa \zeta_{\text{eq}} + 0.05 \quad (3.16)$$

Finally, the elastic spectrum is transformed into a reduced spectrum for the damping ratio  $\zeta_{\text{eq}}$ . The intersection of the capacity and demand in the AD format defines the maximum displacement demand of SDOF which is then transformed back to evaluate the expected response of the building.

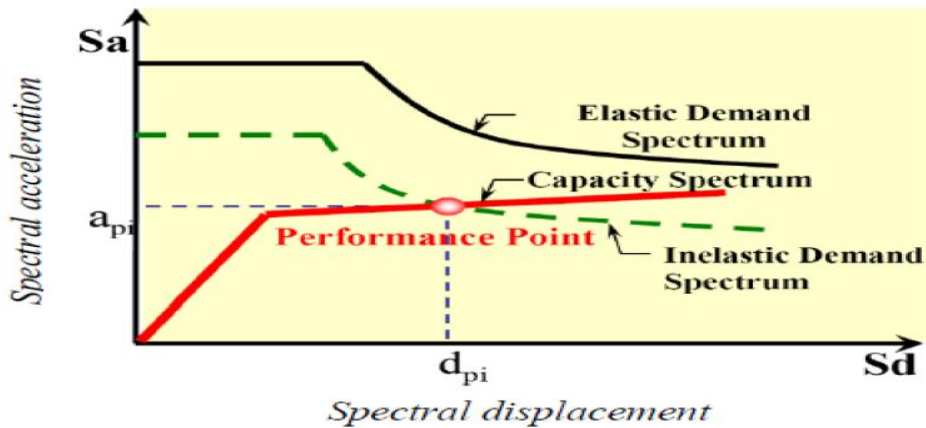
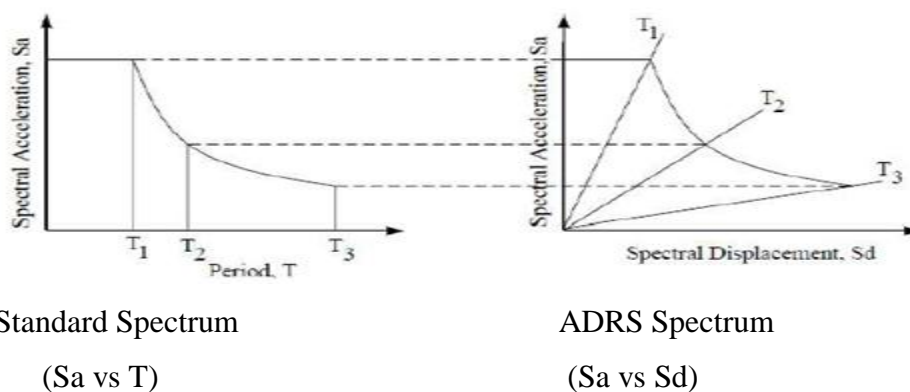


Figure 3.3. Graphical representation of capacity spectrum method [3]

The ADRS format was introduced that the spectral acceleration are plotted against spectral displacements with radial lines representing the period,  $T$ . The demand (inelastic) response spectrum accounting for hysteretic nonlinear behaviour of structure is obtained by reducing elastic response spectrum with spectral reduction factors which depend on effective damping. A performance point that lies on both the capacity spectrum and demand spectrum (reduced for nonlinear effects) is obtained for performance evaluation of the structure. The dependence of spectral reduction factors on structural behaviour type (hysteretic properties) and ground motion duration; and the approximations involved in determination of these characteristics are the main weaknesses of the method, [3].



Standard Spectrum  
(Sa vs T)

ADRS Spectrum  
(Sa vs Sd)

Figure 3.4. Response Spectrum in Standard and ADRS Format [3]

In this paper the response spectrum coefficients are found from the above Table 3.2 of ATC-40 for stiff soil class ( $S_D$ ), which corresponds with soil class B in EBCS-8, 1995 code as:

$$\begin{array}{llll} CA = 0.146 & \text{and } CV = 0.213 & \text{for } PGA = 0.1g, \text{ i.e.} & ZEN = 0.1 \\ CA = 0.08 & \text{and } CV = 0.12 & \text{for } PGA = 0.05g, \text{ i.e.} & ZEN = 0.05 \end{array}$$

### 3.5.2.1.2 Displacement Coefficient Method (DCM)

Displacement Coefficient Method described in FEMA-356 is a non-iterative approximate procedure based on displacement modification factors. The expected maximum inelastic displacement of nonlinear MDOF system is obtained by modifying the elastic spectral displacement of an SDOF system with a series of coefficients. It combines the POA with a modified version of the equal displacement approximation, according to which the linear elastic spectral displacement or the spectral acceleration, corresponding to the effective period and damping of the equivalent SDOF system, is corrected by some factors. These factors were obtained for regular frame buildings. Among its advantages is that the DCM provides a direct numerical procedure to define displacement demand and needs no conversions in spectral format. The target displacement  $\delta_t$  in FEMA-376 is given by:-

$$\delta_t = C_0 C_1 C_2 C_3 S_a T_e^2 / 4\pi^2 g \quad (3.17)$$

Where  $T_e$ , is the effective fundamental period (in seconds) of the building in the direction under consideration,  $S_a$  is the response spectrum acceleration (in g) at the effective fundamental period and damping ratio of the building in the direction under consideration; and  $g$  is the gravity acceleration. To convert elastic demand thus obtained to inelastic demand, the displacement quantity is multiplied by the corrective factors  $C_0$ ,  $C_1$ ,  $C_2$  and  $C_3$ , the minimum prescribed value of these factors being unity. Where factor,  $C_0$  = Modification factor that relates the elastic response of a SDF system to the elastic displacement of the MDF building at the control node,  $C_1$  = Modification factor that relates the maximum inelastic and elastic displacement, stemming from the R- $\mu$ -T relationship it reflects the ratio of the peak displacement of the inelastic system to that of the corresponding elastic system with the same unyielding period of vibration,  $C_2$  = Modification factor to represent the effects of pinched hysteretic shape, stiffness degradation, and strength deterioration, and  $C_3$  = Modification factor to represent increased displacement due to P-delta effects. The major drawback of FEMA 376 is that such a simple produce formulation may not correctly reflect the three distinct failure effects of the actual nonlinear behaviour of structures, [4].

### **3.5.2.1.3 Modal Pushover Analysis (MPA) method**

The results of elastic dynamic analyses of the building can be used to obtain the target displacements and load distributions to pushover analysis of asymmetric buildings ( Tso and Moghadam 1997). This analysis is called response-spectrum-based pushover analysis which takes into account the higher modal and three-dimensional effects induced by torsion. The results of target displacement from of RSA can be multiplied by inelastic deformation ratio to obtain the target displacement for modal pushover analysis method.

The computational effort involved in MPA including the first few two or three modes is comparable to that required in FEMA procedures using two or three lateral-force distributions. With the roof displacement determined from the elastic design spectrum and empirical equations for the ratio of peak deformations of inelastic systems, pushover analysis for each mode requires computational effort similar to one FEMA force distribution, [7].

### **3.6. Seismic Demand, Target Displacement and Performance Point**

The seismic demand on a structure is usually expressed in the form of a design spectrum according to the prevailing seismic code and including all structure and is obtained iteratively. The interaction of the demand spectrum with the nonlinear pushover response is called “Performance Point” or “Target displacement”. It corresponds to the state the structure is expected to reach under the considered earthquake. Depending on the position and state of the performance point (with respect to the actual pushover curve), the analyst may decide on how safe or vulnerable the structure is and where possible strengthening should be performed, [2].

### **3.6 Determination of Target Displacement**

The fundamental question in the execution of the pushover analysis is the magnitude of the target displacement at which seismic performance evaluation of the structure is to be performed. The target displacement serves as an estimate of the global displacement of the structure is expected to experience in a design earthquake. It is the roof displacement at the center of mass of the structure. The extent of damage experienced by the structure at this target displacement is considered representative of the damage experienced by the building when subjected to design level ground shaking. In the pushover analysis it is assumed that the target displacement for the MDOF structure can be estimated as the displacement demand for the corresponding equivalent SDOF system transformed to the SDOF domain through the use of a shape factor. This assumption, which is always an approximation, can only be accepted within limitations and only if great care is taken in incorporating in the predicted SDOF displacement demand all the important ground motion and structural response characteristics

that significantly affect the maximum displacement of the MDOF structure. Inherent in this approach is the assumption that the maximum MDOF displacement is controlled by a single shape factor without regards to higher mode effects. Under the Non-linear Static Procedure, a model directly incorporating inelastic material response is displaced to a target displacement, and resulting internal deformations and forces are determined. The mathematical model of the building is subjected to monotonically increasing lateral forces or displacement is intended to represent the maximum displacement likely to be experienced during the design earthquake, [2].

The target displacement is determined from the elastic response spectrum in Annex A of EBCS-8, 1995 based on a generalized SDOF system equivalence. The target displacement determined in this way is multiplied latter by inelastic deformation ratio to come up with inelastic target displacement.

The method consists of the following steps:

- Transformation of the MDOF system to an equivalent SDOF system.
- Determination of an equivalent idealized elasto-perfectly plastic system.
- Determination of the target displacement for the equivalent system.
- Transformation of the target the MDOF system.
- Multiplying by inelastic deformation.

In modal pushover analysis procedure the seismic demands due to individual terms in the model expansion of the effective earthquake forces are determined by a pushover analysis using the inertia force distributions associated with each mode up to a “modal” target displacement.

The target roof displacement in all of these pushover procedures is determined from the peak deformation of an inelastic single-degree-of-freedom system with its force-deformation relation defined from the pushover curve.

### **3.7 Performance objectives**

A performance objective specifies the desired seismic performance of the building. Seismic performance is described by designating the maximum allowable damage states (performance level for an identified seismic hazard (earthquake ground motion)). A performance objective may include consideration of damage states for several level of ground motion and would then be termed a dual-or multiple-level performance objective, [3].

### 3.8 Performance levels

A performance level describes a limiting damage condition which may be considered satisfactory for a given ground motion.

The limiting condition is described by the physical damage within the building, the threat to life safety of the buildings occupants create by the damage, and the post-earthquake serviceability of the building.

Target performance levels for structure and non structure systems are specified independently. Structural performance levels are given names and letter designations. Building performance are a combination of a structure performance and a non structural performance level and are designated by the application number and letter combination such as 1-A, 3-C, etc,[3].

#### 3.8.1 Structural Performance Levels and Ranges

Structural performance levels and range are assigned a title and, for case of reference, a number. The number is called the structural performance number and abbreviated Sp-n (where n is the designation number).

The structural performance level-Immediate Occupancy, Life Safety, and Structural Stability-are discrete damage states and can be used directly in evaluation and retrofit procedures to define technical criteria. The other structural performance designations. Damage Control, Limited Safety and Not Considered are important place holders in the numbering scheme to allow direct reference to the wide variety of building performance levels that might be desirable to owner for evaluation or retrofit, [3].

- **Immediate Occupancy, SP-1:** The post-earthquake damage state in which only very limited structural damage has occurred. The basic vertical and lateral force resisting systems of the building retain nearly all of their pre-earthquake characteristics and capacities. The risk of life-threatening injury from structural failure is negligible, and the building should be safe for unlimited egress, ingress, and occupancy.
- **Damage Control, SP-2:** This term is actually not a specific level but a range of post-earthquake damage states that could vary from SP-1, Immediate Occupancy to SP-3, Life Safety. It provides a placeholder for the many situations where it may be desirable to limit structural damage beyond the Life Safety level, but Occupancy is not the issue. Examples of damage control include protection of significant architectural features of historic buildings or valuable contents.
- **Life Safety, SP-3:** The post-earthquake damage states in which significant damage states in which significant damage to the structure may have occurred but in which some margin

against either total or practical structural collapse remains. The level of damage is lower than that for the structural stability level. Major structural components have not become dislodged and fallen, threatening Life Safety either within or outside the building. While injuries during the earthquake may occur, the risk of life-threatening injury from structural repairs will likely be necessary prior to reoccupation of the building, although the damage may not always be economically repairable. This level of structural performance is intended to be less than the level of performance expected of fully code compliant new buildings.

- **Limited Safety, SP-4** : This term is actually not specific level but a range of post-earthquake damage states that are less than SP-3, Life Safety and better than SP-5, structural stability. It provides a place holder for the situation where retrofit may not meet all the structural requirements of the Life Safety level, but is better than the level of structural stability. These circumstances include cases when the complete Life Safety level is not cost effective, or when only some critical structural deficiencies are mitigated. (The non-structural performance level used in this range varies and will depend on the intent of the damage control).

**Structural Stability, SP-5:** This level is the limiting post-earthquake structural damage states in which the buildings structural system is on the verge of experiencing 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. However, all significant components of the gravity demands. Although the building retains its overall stability, significant risk of injury due to falling hazards may exist both within and outside the building and significant major structural repair will be necessary prior to preoccupation. In the other concrete building types also considered.

Falling hazards are not significant are not significantly prevented to achieve this performance not considered is normally combined with SP-5.

### **3.8.2 Non structural performance Levels**

Non structural performance levels are assigned a title and, for ease of reference, a letter. The letter is called the non structural performance letter and is abbreviated NP-n (where n is the designated letter).

The non structural performance levels-Operational, Immediate Occupancy, Life Safety, and Hazards Reduced – are discrete damage states and can be used directly in evaluation and retrofit procedures to define technical criteria. The other non structural performance designation –Not considered –is an important placeholder to allow direct reference to the

wide variety of building performance levels that might be desirable to owners for evaluation or retrofit, [3].

- **Operational, NP-A:** The post-earthquake damage state in which non structural elements and systems are generally in place and functional, Although minor disruption and cleanup should be expected, all equipment and machinery should be working. However, external utilities, which may not be available due to significant off-site damage, must be locally backed up. Contingency plans to deal with possible difficulties with external communication, transportation, and availability of supplies should be in place.
- **Immediate Occupancy, NP-B:** The post-earthquake damage state in which non structural elements and systems are generally in place. Minor disruption and cleanup should be expected, particularly due to damage or shifting of contents. Although equipment and machinery are generally anchored or braced, their ability to function after strong shaking is not considered and some limitations on use or functionality may exist. All external utilities may not be locally backed up. Seismic safety status should not be affected.
- **Life Safety, NP-C:** This post-earthquake damage state could include considerable damage to non structural components and systems but should not include collapse or falling of items heavy enough to cause severe injuries either within or outside the building. Secondary hazards from breaks in high-pressure, toxic, or fire suppression piping should not be present. Non structure systems, equipment, and machinery may not be functional without replacement or repair. While injuries during the earthquake may occur, the risk of life-threatening injury from non structure damage in very low.
- **Reduced Hazard, NP-D:** This post-earthquake damage state could include extensive damage to non structure include collapse or falling of large and heavy items that could cause significant injury to groups of people, such as parapets, masonry exterior walls, cladding, or large, heavy ceilings. While isolated serious injury could occur, risk of failures that could put large numbers of people at risk within or outside the building is very low.
- **Not Considered, NP-E:** Non structural elements, other than those that have an effect on structural response, are not evaluated.

### 3.8.3 Building Performance Levels

Combinations of a structure performance level and a non structural performance level to completely describe the desired limiting damage state for a building. The four most

commonly referenced building performance levels are given titles and are described below, [3].

- **Operational, 1-A:** This is the performance level related to functionality. Damage to the buildings structure is limited so that continued safe occupancy is not question, and any required repairs are minor and can be carried out side without significant disruption to occupants. Similarly, damage to non-structural systems and contents related to functionality is minor and will not jeopardize functions in the building. Most importantly, vital services from outside the building such as utilities, transportation, or communications must be provided with back-up facilities or planning as required to allow functions to continue if these services are unavailable. Since important aspects of this performance objective involve contingency planning and design of back-up systems.
- **Immediate Occupancy, 1-B:** This corresponds to the most widely used criteria for essential facilities. The buildings spaces and systems are expected to be reasonably usable, but continuity is expected to be reasonably usable, but continuity of all services, either primary or backup, is not necessarily provided. Contents may be damaged.
- **Life Safety, 3-C:** This level is intended to achieve a damage state that presents an extremely low probability of threats to life safety, either from structural building components. User-furnished contents, however, are not controlled, and could create falling hazards or secondary hazards, such as chemical releases or fire. This performance level is intended to be less than the performance that is expected of code designed new buildings.
- **Structural Stability, 5-E:** This damage state addresses only the main building frame or vertical load carrying system and requires only stability under vertical loads. No margin against collapse in aftershocks may be available. Life threatening external or internal falling hazards from cladding, non-structural finishes, or even structural damage may have occurred. Review of performance of non-structural element from expected forces or structural drifts is not required so their performance can be highly unreliable.

#### **3.8.4 Other Commonly Used Combinations**

- **Building Performance Level 3-D:** This level combines life safety structural performance with the reduced hazard non-structural performance, thus accepting a slight risk to life safety from non-structural systems. Although large and highly vulnerable non-structural elements such as mechanical/electrical equipment and distribution systems, partitions, and typical ceilings and light fixtures have not been braced or anchored and could be highly disrupted and produce falling hazards.[3]

- **Building Performance Level 3-B:** This level presents a risk of structural damage that could prevent the building from being occupied. However, nearly complete non-structural protection will prevent significant internal disruption, particularly in low levels of shaking. Although only seldom applied to a whole building, this level is more commonly applied to particular areas or rooms, such as computer facilities.[3]

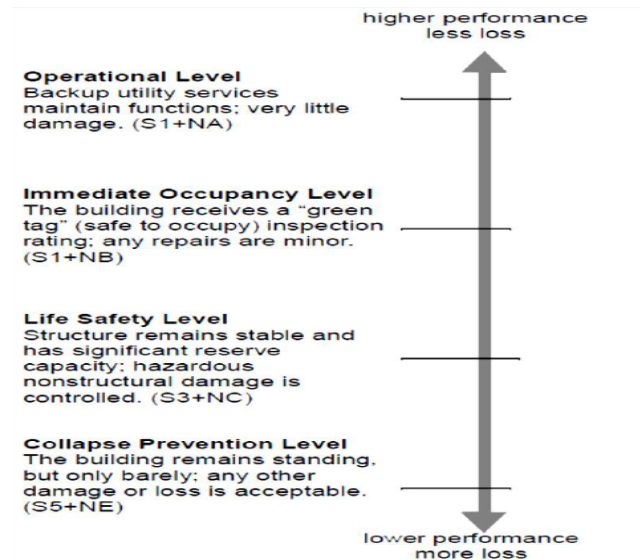


Figure 3.5 Building performance levels [20]

### 3.9 Earthquake Ground Motion

Earthquake ground motion is combined with a desired performance level to form a performance objective. This earthquake ground motion can be expressed either by specifying a level of shaking associated with a given probability of occurrence (a probabilistic approach). Or in terms of the maximum shaking expected from a single event of a specified magnitude on a specified source fault (a deterministic approach). The level of ground motion is expressed in terms of engineering characteristics for use in design. A response spectra or an equivalent series of simulated recordings of earthquake motions are used for this purpose.

The following three levels of earthquake ground motion are defined as follow, [3].

- The Serviceability Earthquake (SE): Ground motion with a 50 percent chance of being exceeded in a 50-year period.
- The Design Earthquake (DE): Ground motion with a 10 percent chance of being exceeded in a 50-year period.
- The Maximum Earthquake (ME): Maximum level of ground motion expected within the known geologic framework due to a specified single event (median attenuation), or the ground motion with a 5 percent chance of being exceeded in a 50-year period.

## CHAPTER FOUR-MODELING OF STRUCTURAL ELEMENT

### 4.0 General

For complex model pushover analysis is numerically demanding and may cause numerical difficulties for the software used to run the analysis. So it is advisable simplifying the model as much as possible in order to be helpful in completing the runs and reducing the running time. As the analysis process may usually involve several runs to evaluate effects of the various parameters, it is important to be able to complete a pushover run in less time. Modelling with least possible amount of meshing for any linear element should be recommended. Hinges should be assigned to any location where non linear behaviour is expected, even when non linear behaviour is later not observed at some of the locations. Having more hinges than necessary does not slow down analysis and it ensures that non linear behaviour is capture. The most important step for the entire analysis is identification of the primary structural elements, which must be completely modelled in the analysis. Secondary elements do not need to be included in the analysis because they do not significantly contribute to the buildings lateral forces resisting system. A model with some elements that yield much earlier compared to the rest may be numerically difficult to run. This may happen when elements that are not the main components of lateral system are modelled with hinge. As these elements are expected to yield early in an earth quake, an easy solution may be model them as pin ended. Moreover, building behaviour should be verified to not have changed significantly in this process. This may be done comparing the capacity curve for the model with pin-ended element to that of the original model. However, this element must be detailed to the yield in a ductile manner.[2]

Rigged end offsets significantly influence force distribution between elements and model behaviour. In shear wall buildings where pushover model uses frame elements to model shear walls, the clear span of spandrels and any slender columns formed due to wall opening is usually much smaller than the centre to centre span. These elements should be modelled with rigged end offsets and non linear hinges must be assigned outside of the offset.[2]

### 4.1 Types of inelastic structural analysis models

Inelastic structural component models can be differentiated by the way that plasticity is distributed along its length and through the member cross-section. Figure 4-1 below shows a comparison of five idealized model types for simulating the inelastic response of beam-columns. Several types of structural member (e.g. beam, column, braces, and some flexural walls) can be modelled using the concept illustrated in figure 4-1

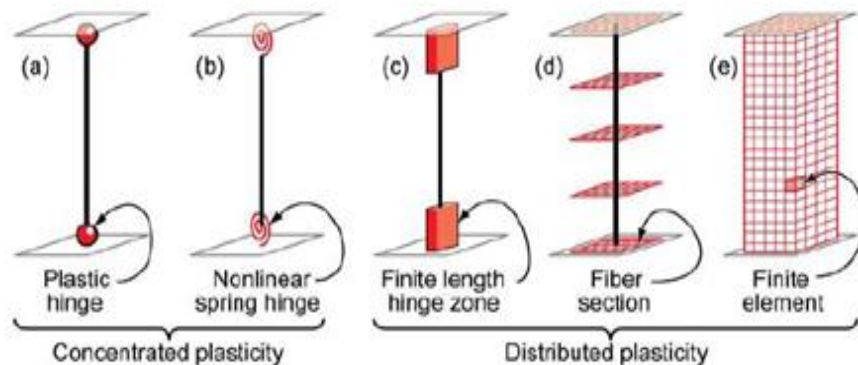


Figure 4.1. Types of inelastic structural models from NEHRP Seismic Design

The simplest models concentrate the inelastic deformations at the end of the element, such as through a rigid-plastic hinges (Figure 4-1a) or an inelastic spring with hysteretic properties (Figure 4-1b). By concentrating the plasticity in zero-length hinges with moment-rotation model parameters, these elements have relatively condensed numerically efficient formulations.

The finite length hinge model (Figure 4.1c) is an efficient distributed plasticity formulation with designed hinge zones at the member ends. Cross-section in the inelastic hinge zones are characterized through either nonlinear moment-curvature relationships or explicit fibre-section integration that enforce the assumption that plane section remain plane. The inelastic hinge length may be fixed or variable as determined from moment-curvature characteristics of the section together with the concurrent moment gradient and axial force. Integration of deformations along the hinge length captures the spread of yielding more realistically than the concentrated hinges, while the finite hinge lengths facilitate calculation of hinge rotation.

The fiber formulation (Figure 4-1d) models distributed plasticity by numerical integrations through the member cross sections and along the member length. Uniaxial material models are defined to capture the nonlinear hysteretic axial stress-strain characteristics in the cross sections. The plane-sections-remain-plane assumption is enforced, where uniaxial material “Fibers” are numerically integrated over the cross section to obtain stress resultants (axial force and moments) and incremental moment-curvature and axial force-strain relations. The cross section parameters are then integrated numerically at discrete sections along the member length, using displacement or force interpolation functions (Kunnath et al. 1990, Spacone et al. 1996). Distributed fibre formulations do not generally report plastic hinge rotations, but instead report strains in the steel and concrete cross section fibres. The

calculated strain demands can be quite sensitive to the moment gradient, element length, integration method, and strain hardening parameters on the calculated strain demands. Therefore, the strain demands and acceptance criteria should be benchmarked against concentrated hinge models, for which rotation acceptance criteria are more widely reported. The most complex models (Figure 4.1e) discretize the continuum along the member length and through the cross sections into small (micro) finite elements with nonlinear hysteretic constitutive properties that have numerous input parameters. This fundamental level of modelling offers the most versatility, but it also presents the most challenge in terms of model parameter calibration and computational resources. As with the fibre formulation, the strains calculated from the finite elements can be difficult to interpret relative to acceptance criteria that are typically reported in terms of hinge rotations and deformations. Concentrated and finite length hinge models (Figures 4.1a through Figure 4.1c) may consider the axial force-moment (P-M) interactions through yield surfaces. On the other hand, fibre (Figure 4.1d) and finite element (Figure 4.1e) models capture the P-M response directly. Note that while the detailed fibre and finite element models can simulate certain behaviour more fundamentally, they are not necessarily capable of modelling other effects, such as degradation due to reinforcing bar buckling and fractures that can be captured by simpler phenomenological models interpreted relative to acceptance criteria that are typically reported in terms of hinge rotations and deformations.[2]

Some types of concentrated hinge models employ axial load-moment (P-M) yield surfaces. Whereas these models generally do a good job at tracking the initiation of yielding under axial load and bending, they may not capture accurately the post-yield and degrading response. On the other hand, some hinge elements with detailed moment-rotation hysteresis models may not capture P-M interaction, except to the extent that the moment-rotation response is defined based on average values of axial load and shear that are assumed to be present in the hinge, [2].

#### **4.1.1 Distributed Versus Concentrated Plastic Hinge**

While distributed plasticity formulations (Figures 4.1c through 4.1e) model variations of the stress and strain through the section and along the member in more detail, important local behaviours, such as strength degradation due to local buckling of steel reinforcing bars or flanges, or the nonlinear interaction of flexural and shear, are difficult to capture without, sophisticated and numerically intensive models. On the other hand, phenomenological

Concentrated hinge/spring models (Figure 4.1a and 4.1b), may be better suited to capturing the nonlinear degrading response of members through calibration using member test data on phenomenological moment-rotations and hysteresis curves. Thus, when selecting analysis model types, it is important to understand (1) the expected behaviour, (2) the assumptions, and (3) the approximations inherent to the proposed model type. While more sophisticated formulations may seem to offer better capabilities for modelling certain aspects of behaviour, simplified models may capture more effectively the relevant feature with the same or lower approximation. It is best to gain knowledge and confidence in specific models and software implementations by analyzing small test examples, where one can interrogate specific behavioural effects.

In concentrated plasticity approach, the effect of material yielding is “lumped” into a dimensionless plastic hinges (Figure 4-1a). Regions in the frame elements other than at the plastic hinges are assumed to behave elastically, and if the cross-section forces are less than cross-section plastic capacity, elastic behaviour is assumed. When the steady-forces reach the yield surface, a plastic hinge is formed which follow the no hardening plasticity flow rules. To develop the incremental elasto-plastic relations, following standard practices of the non hardening plasticity flow theory, the incremental elasto-plastic stiffness matrix can be generated.

The plastic hinge approach eliminates the integration process on the cross section and permits the use of fewer elements for each member, and hence greatly reduces the computing effort. However, the method has been shown to overestimate the limit load in the case of reinforced concrete structures, where spread of plasticity effects is very significant.[2]

#### **4.1.2 Frame Hinge Properties**

The capacity of providing plastic hinges as discrete user defined hinges along the clear length of the frame element introduced by SAP 2000/ETABS. And also the post-yield behaviour in one or more degree of freedom represented by plastic hinge. Uncoupled moment, torsion, axial force and shear hinge are available to be modelled along the frame element .Also, a P-M2-M3 hinge which yields based on the interaction of axial force and bending moments at the hinge location can be modelled. Sometime more than one type of hinge can exist at the same location, for example, the user might assign both M3 (moment) and V (shear) hinge to the same end of a frame element. In the analytical modelling used in SAP 2000 /ETABS software the hysteretic response of the concentrated plasticity at the ends of a member can be described by a moment curvature relationship. The program can specify for each material one

or more stress- strain curves that are used to generate nonlinear hinge properties in frame elements. The different curves can be used for different parts of frame cross-section.

For nonlinear analysis user-defined hinge properties and automatic hinge properties can be assigned to frame element. When user-defined or automatic hinge properties are assigned to frame element, the program automatically creates a generated hinge property for each and every hinges. User-defined hinge property can either be based on a hinge property generated from automatic, or they can be fully user-defined. A generated property can be converted to user-defined and then modified and re-assigned to one or more frame elements. Automatic hinge properties are based upon a simplified set of assumptions that may not be appropriate for hinge all structure. If someone want to use automatic properties as starting points, and then convert the corresponding generated hinges values as needed.

The main reason for differentiation between generated properties and defined (automatic and user-defined) properties is that typically the hinge properties are section dependent. Thus it would be necessary mean that someone would need to define a large number of hinge properties. The definition of user-defined hinge properties requires moment- curvature analysis of each of each element. For a particular axial force, moment-rotation ( $M-\theta$ ) characteristic of a frame member with lumped plasticity gives a measure of rotation ductility capacity of the member.[9] and [13]

In SAP2000, the default-hinge model assumes the same deformation capacity for all columns regardless of their axial load and their weak and strong axis orientation. It takes the average values of hinge properties instead of carrying out detailed calculation for each member. But, the hinge properties depend on the type of element, material property, shear span ratio and the axial load on the element. To account for this, in the present study, user-defined hinge properties obtained from the yield, plastic and ultimate rotation characteristics ( $\theta_y$ ,  $\theta_p$ ,  $\theta_{ult}$ ) of typical elements are estimated. Using this method, the inelastic hinge effects of beams and columns of the buildings are analysed. The force-deformation behaviour of hinges such as IO, LS and CP are defined and also incorporated in the software. The input required for SAP2000/ETABS is moment rotation relationship instead of moment-curvature. Also, moment rotation data have been reduced to five-point input that brings some inevitable simplifications. Plastic hinge length is used to obtain ultimate rotation values from the ultimate curvatures. Several plastic hinge lengths have been proposed in the literature (Park and Paulay, 1975; Priestley et al, 1996). In this study plastic hinge length definition given in Eqn. 4.2 which is proposed by (Priestley et al, 1996) is used.

$$L_p = 0.08L_i + 0.022 f_{yh} \phi_{bl} \geq 0.044 f_{yh} \phi_{bl} \quad (4.1)$$

Where:  $L_p$  = the plastic hinge length

$L_i$  = the distance from critical section of the plastic hinge to the point of contra flexure

$\phi_{bl}$  = the diameter of longitudinal reinforcement

$f_{yh}$  = the yield strength of transverse reinforcement

In existing reinforced concrete buildings, especially with low concrete strength and/or insufficient amount of transverse steel, shear failures of members should be taken into consideration. For this purpose, shear hinges are introduced for beams and columns. Because of brittle failure of concrete in shear, no ductility is considered for this type of hinges. Shear hinge properties are defined such that when the shear force in the member reaches its strength, member fails immediately.

The moment is assumed to vary linearly along the beams and columns with a contra flexure point at the middle of the members. Based on this assumption, the relationship between curvature and rotation at yield is obtained as follows;

$$\theta_y = L \cdot \phi_y / 6 \quad (4.2)$$

Where  $L$  = member length

$\phi_y$  = Curvature at yield

$\theta_y$  = Rotation at yield

$$\theta_p = (\phi_{ult} - \phi_y) l_p \quad (4.3)$$

Where  $l_p$  = plastic hinge length

$\phi_{ult}$  = ultimate curvature

$\theta_p$  = plastic Rotation

Rotation value at ultimate moment is obtained by adding plastic rotation to the yield rotation.

In the models considered for this study, the moment-curvature response of the cross-section is determined and transformed in to moment-rotation. The results are then used as an input to change the generated hinges to user defined hinge properties option in SAP2000/ETABS.

### 4.1.3 Plastic Hinge Definition

The plastic hinge zone to be used is affected by different parameters like the compressive strain. Different researchers gives different values, If we use the Paulay and Priestley (1992),

$LP = 0.08Z + 0.0022dbfy$  (for RC beams and Columns) we will different values depending on the member length, bar size and contra flexure points. Mostly ,for SAP2000/ETABS a hinge point with 0 strain hardening is used so there is no plastic zone.

Figure 4.2 shows the hinge properties definition for beams and columns on SAP2000 using the moment curvature relation obtained from response 2000.

- Beam 20x50 section with reinforcement of 6 $\phi$ 20( taken as sample).

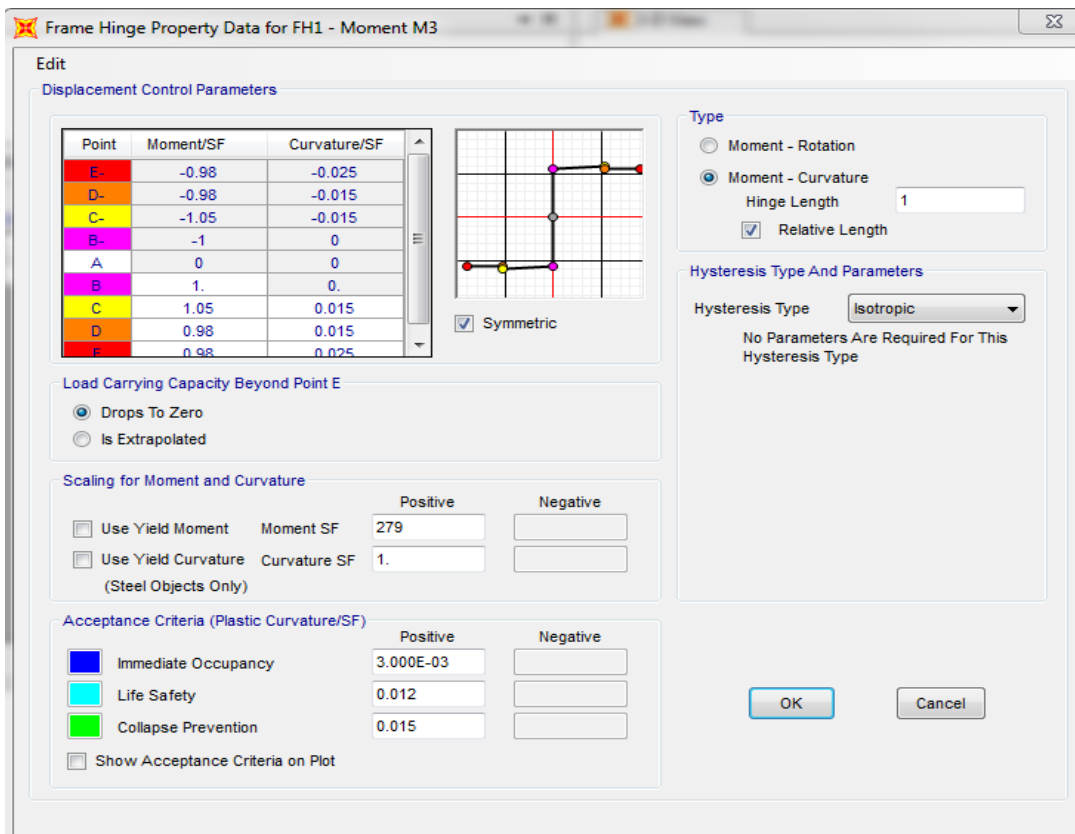


Figure 4.2 Hinge Definition on SAP2000

## 4.2 Modelling of Structural Components

### 4.2.1 Material Nonlinearity

#### Column and Beams

Column-beam are commonly modelled using either fiber-type element or concentrated hinges. While the fiber elements generally enable more accurate modelling of the initiation of inelastic effects( concrete cracking and steel yielding) and spread of yielding , their ability may be limited to capture degradation associated with bond slip in concrete joints and local buckling and fracture of steel reinforcing bars and steel members, [2].

## **Shear Walls**

Reinforced concrete shear walls are commonly employed in seismic lateral-force-resisting system for buildings. They may take the form of isolated planar walls, flanged walls (often T-C or T-shaped in plans) and larger three dimensional assemblies such as building cores. Nearly walls are often connected by coupling beams from greater structural efficiency where larger openings for doorways are required. The seismic behaviour of shear walls is often distinguished between squat (shear governed) and slender (ductile flexure governed) according to the governing mode of yielding and failure. In general, it is desirable to achieve ductile flexural behaviour, but this is not possible in circumstances such as (1) in existing buildings without seismic design and detailing, (2) bearing walls with high axial stress and (or inadequate confinement that are susceptible to compression failures, and (3) short walls with high shear-to-flexure ratios that are susceptible to shear failures.

Slender concrete shear walls detailed to current seismic design requirements, having low axial stress, and designed with sufficient shear strength to avoid shear failures, perform in a similar manner to reinforced concrete beam-columns. Ductile flexural behaviour with stable hysteresis can develop up to hinge rotation limits that are a function of axial load and shear in the hinge region. Simple slender walls (including coupled walls) can be modelled as vertical beam-column elements with lumped flexural plastic hinges at the ends with reasonable accuracy and computational efficiency. The modelling parameters and plastic rotation limits of ASCE 41 may be used for guidance. The following points should be noted: The lumped hinge models are only suitable for assessing performance within such allowable plastic hinge rotation limits as stable hysteresis occurs, considering axial and shear forces in the hinge.

Nonlinearity only arises at the designated hinge(s), and equivalent flexural and shear stiffness must be specified for elastic elements outside of the hinge. ASCE 41 provides guidance on effective stiffness parameters that account for flexural and shear cracking to handle typical cases (i.e., planar walls with typical reinforcement, wall proportions, and gravity stresses).

Beam-column elements are more problematic to use in three-dimensional wall configurations With significant bi-directional interaction, particularly if the wall system is subjected to torsion, [2].

## **Modelling of Shear Walls as Equivalent frame Method**

In modelling shear walls, each planar wall in the assembly is replaced with a column having the same mechanical properties of the wall as in the equivalent frame method. In order to

ensure the vertical compatibility of the displacements, the rigid beams at floor levels are rigidly connected to each other at the corners. In addition, the ends of the rigid beams that are connected to each other are released (disconnected) from the connection joint only for torsional moments. In another words, the transfer of torsional moments between rigid beams is prevented. The rigid link elements used to connect the beams to the wall element are modelled as rigid beam with end offset properties activated along the entire length of the element, which implies so high values for the section stiffness that the beam can be assumed as fully rigid. In Figure 4.2, the connection details of orthogonal shear walls are given. In three dimensional analyses of shear wall assemblies modelled by the conventional equivalent frame model, serious errors occur especially in the analysis of assemblies subjected to torsion. The stiffness of the structural system becomes stiffer than with finite element modelling. Releasing the ends of the rigid beams from the connection joint decreases the torsional stiffness of the shear wall assembly.

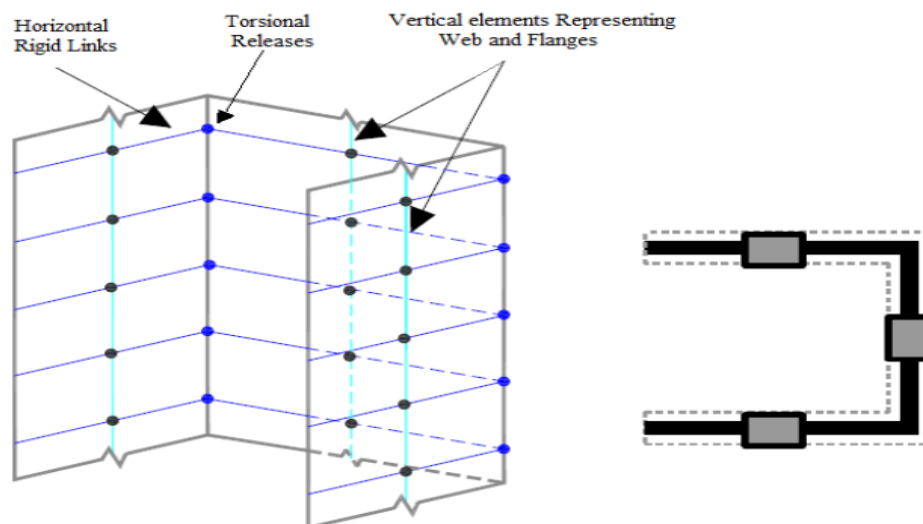


Figure 4.3. Model scheme used to represent U-shaped wall system (Beyer et al.2008)

Given that all shear walls in the building are slender with wall height-to-length ratio well above three and therefore seismic response of the shear walls is expected to be dominated by flexure, as well as because modelling nonlinear behaviour in SAP2000/ETABS pushover analysis is limited to frame elements, the shear walls were modelled as equivalent frame elements. In order to provide connectivity between walls, the equivalent frames were connected at the floor level with rigid links on the side of the wall without any opening, or

with beams with rigid end offsets to model spandrels above wall openings. Figure 4.3 illustrates this modelling technique for the longitudinal walls in the middle core, [2].

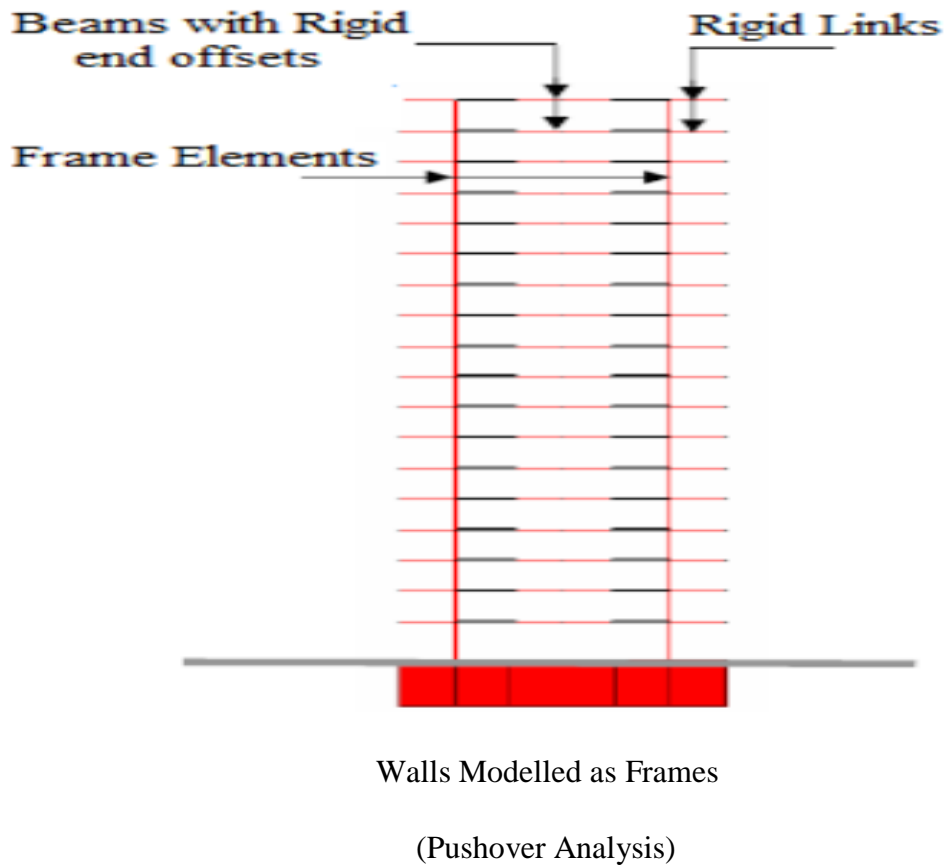


Figure 4.4. Shear wall modelling in pushover analysis (Rane et al. 2004)[22]

### Nonlinear Behaviour of Structural Elements

The nonlinear behaviour of the building structure depending on the nonlinear responses of the elements that are used in the lateral force resisting system. Therefore, before applying any nonlinear analysis method on a building structure, the nonlinear behaviour of such elements must be clearly described and evaluated.

ATC-40 and FEMA-356 codes define the acceptance criteria depending on the plastic hinge rotations by considering various performance levels. In figure 4.4. The five points (A, B, C, D and E) which are used to define the hinge rotation behaviour of RC members and the acceptance criteria on a force versus deformation diagram are given. In this diagram, points marked as IO, LS and CP represent Immediate Occupancy, Life Safety and Collapse Prevention, respectively.

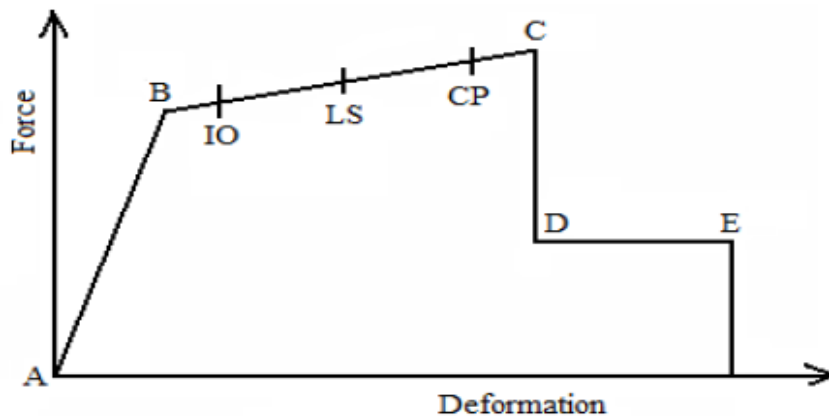


Figure 4.5. Acceptance Criteria on a force versus deformation diagram.

In this study, the hinge properties are determined according to FEMA-356[25]

#### 4.2.2. Geometric Nonlinearity

Geometric nonlinearity is the change in the elastic load-deformation characteristics of the structure caused by the change in the structural shape due to large deformations. It is caused by gravity loads acting on the deformed configuration of the structure, leading to an increase of internal forces in members and connections. It plays a fundamental role in the global response of the structure when the occurrences of large deformation in the structural elements induce displacements not more proportional to the loads effectively applied. Involving both local and global aspects, there are three most important sources of geometric nonlinearities: the beam column effects, the large displacement/rotation effects and the P-delta effects. These geometric nonlinear effects are typically distinguished between P- $\delta$  effects, associated with deformations along the members, measured relative to the member chord, and P- $\Delta$  effects, measured between member ends and commonly associated with story drifts in buildings. In buildings subjected to earthquakes, P- $\Delta$  effects are much more of a concern than P- $\delta$  effects, and provided that members conform to the slenderness limits for special systems in high seismic regions. P- $\delta$  effects do not generally need to be modelled in nonlinear seismic analysis. On the other hand, P- $\Delta$  effects must be modelled as they can ultimately lead to loss of lateral resistance, ratcheting (a gradual build up of residual deformations under cyclic loading), and dynamic instability. Large lateral deflections ( $\Delta$ ) magnify the internal force and moment demands, causing a decrease in the effective lateral stiffness. With the

increase of internal forces, a smaller proportion of the structure's capacity remains available to sustain lateral loads, leading to a reduction in the  $\Delta$  effective lateral strength. Shown in Figure 4.6 is an idealized base shear versus drift curve of a cantilever structure with and without P- $\Delta$  effects. If the gravity load is large the stiffness reduction is significant and contributes to loss of lateral resistance and instability. Therefore the gravity load-deformation (P- $\Delta$ ) effect must be considered directly in the analysis, whether static or dynamic. This means that the gravity loads of the entire building must be present in the analysis, and appropriate P- $\Delta$  analysis techniques should be introduced in the structural model (Wilson 2002; Powell 2010).

For nonlinear seismic analyses, ASCE 7 specifies a gravity load combination of  $1.0D + 0.5L$ , where D is the building dead load and L is the specified live load, including allowance for live load reduction, [2].

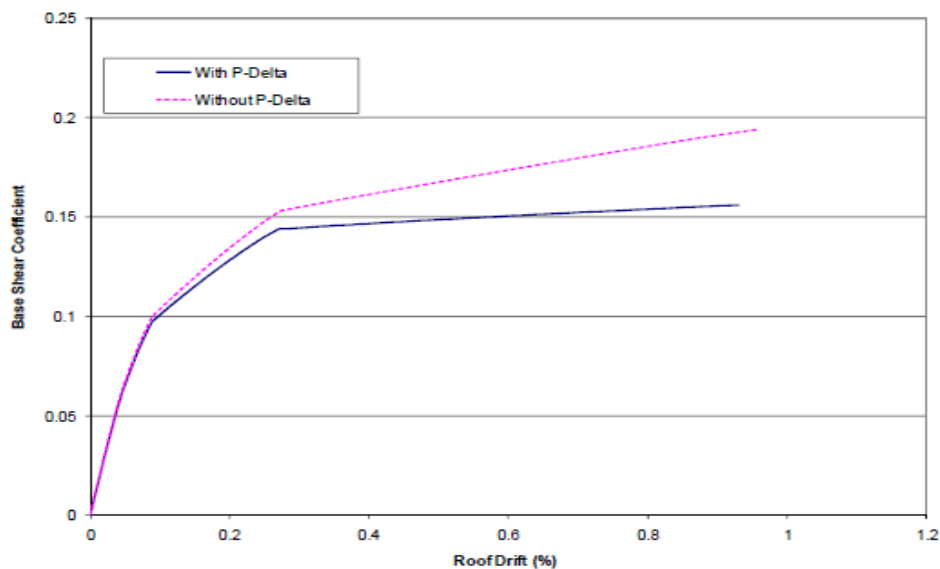


Figure 4.6. Typical Pushover curve with and without P- $\Delta$  effect [2]

### 4.3. Descriptions of the Analyzed Buildings

In this study two buildings as case study are selected namely forty-sixty (G+7 cost-efficient Apartments) uses for living purposes constructed in Addis Ababa. The first is regular rectangular shape with and without shear wall and the second one is irregular or L-shaped with and without shear wall have been taken in order to have better insight for both regular and irregular structure as well as the effects of shear wall on both structure have been checked also.. The structural system used for these building is taken as concrete moment-

resisting space frame (MRSF), and the soil type is considered as class B. The ductility class of the building is taken as “low”, (DL) because most building structure have been design using ductility class low. Furthermore, the design acceleration has been taken as 0.05g which corresponds to that used for low seismic zone (zone 2) in EBCS, 8, 1995 in the designing of the structure first. In order to understand the effect caused by the gravitational acceleration change from 0.05g (old code) to 0.1g (new code) on the performance of existing structure using pushover analysis have been checked first for old code and second for new code( ductility class medium is taken) for both selected apartment including with and also without the present of shear well. And in the modelling of the two structure Euro code is used.

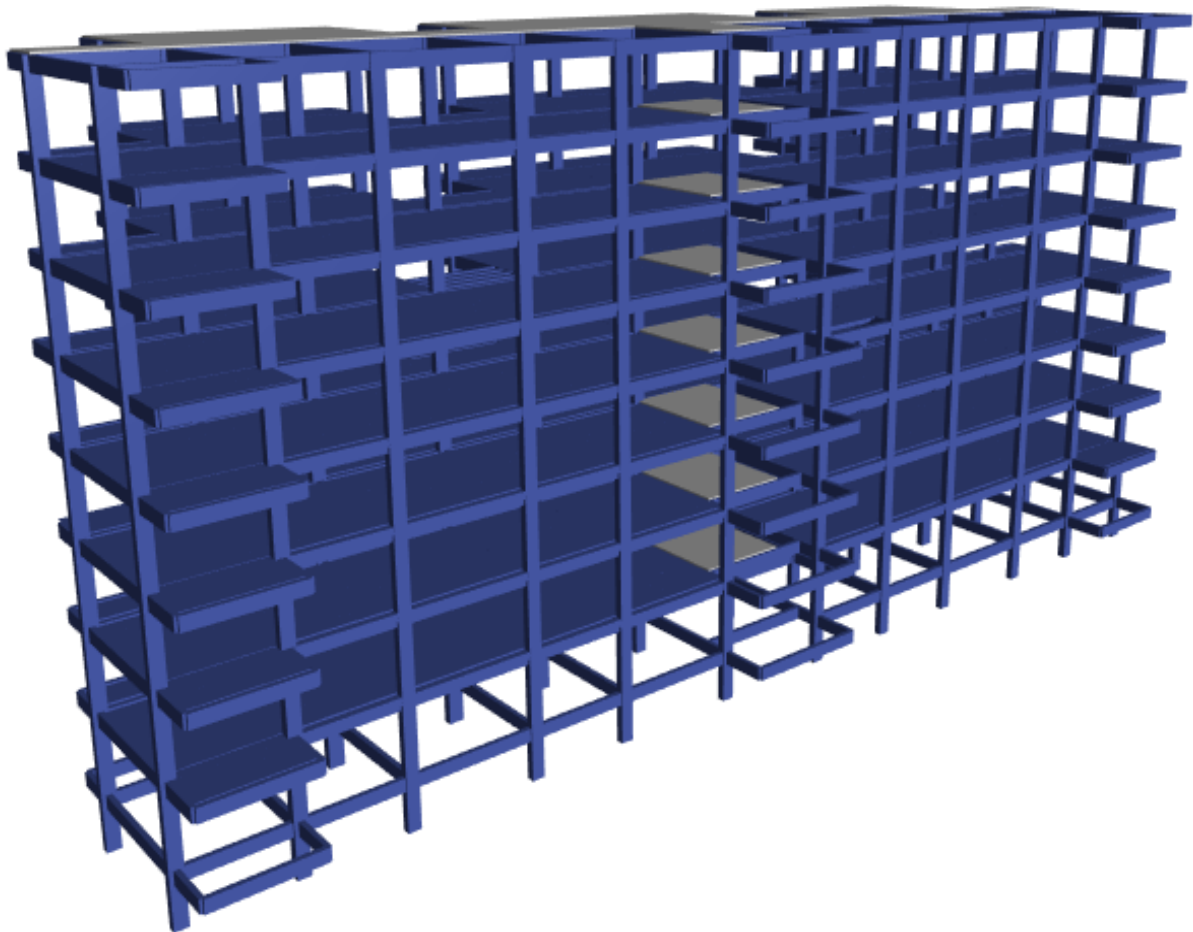


Figure 4.71. Three dimensional view for first bay of Eight Storey RC Building without shear wall

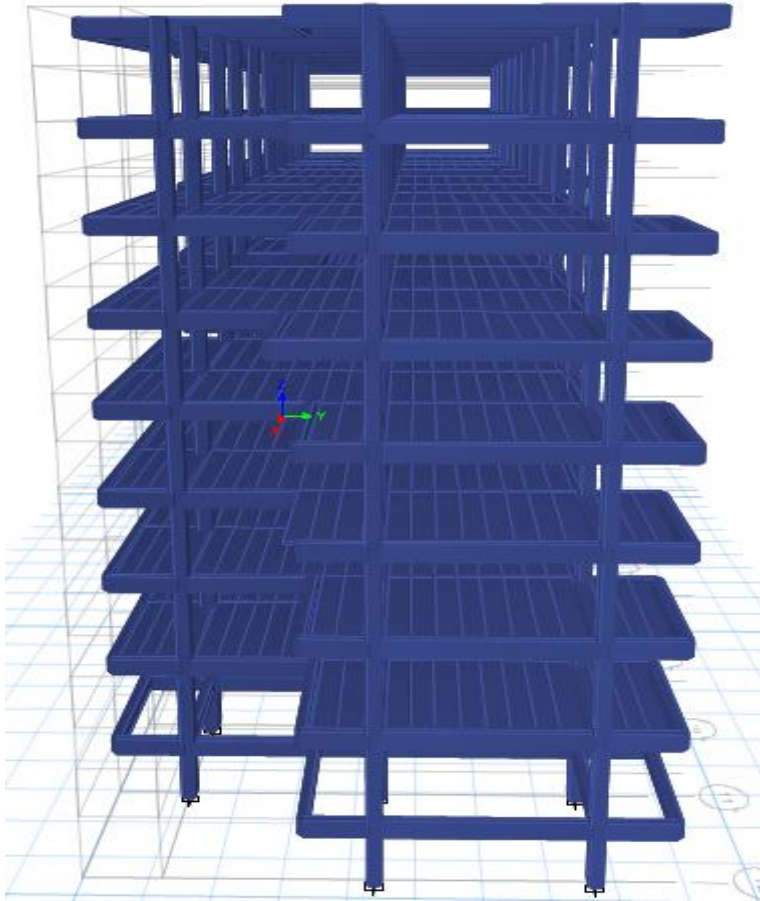


Figure 4.72. Three dimensional views for second bay of Eight Storey RC Building without shear wall

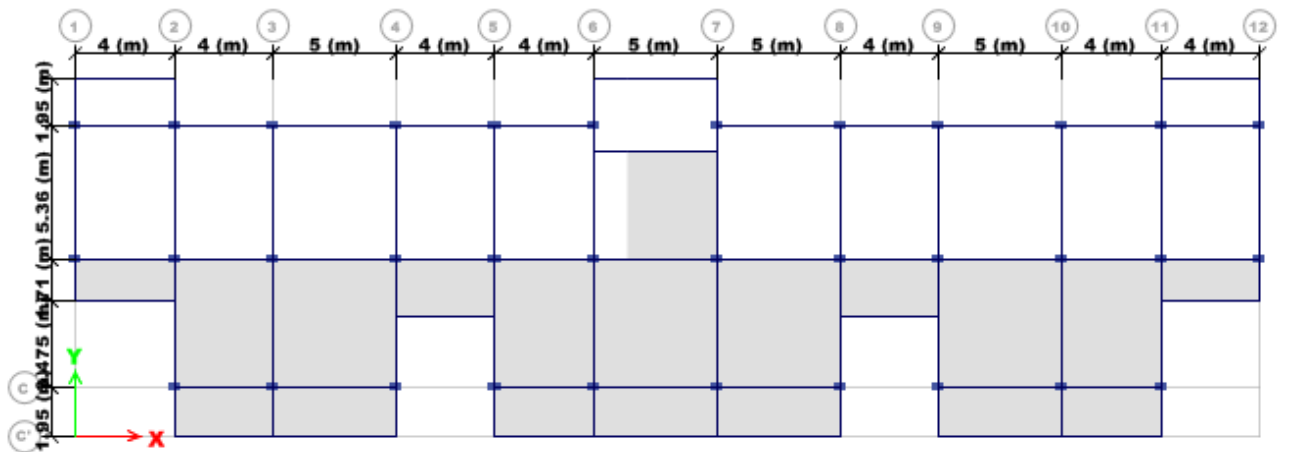


Figure 4.73. Plan view of Eight Storey RC Building without shear wall

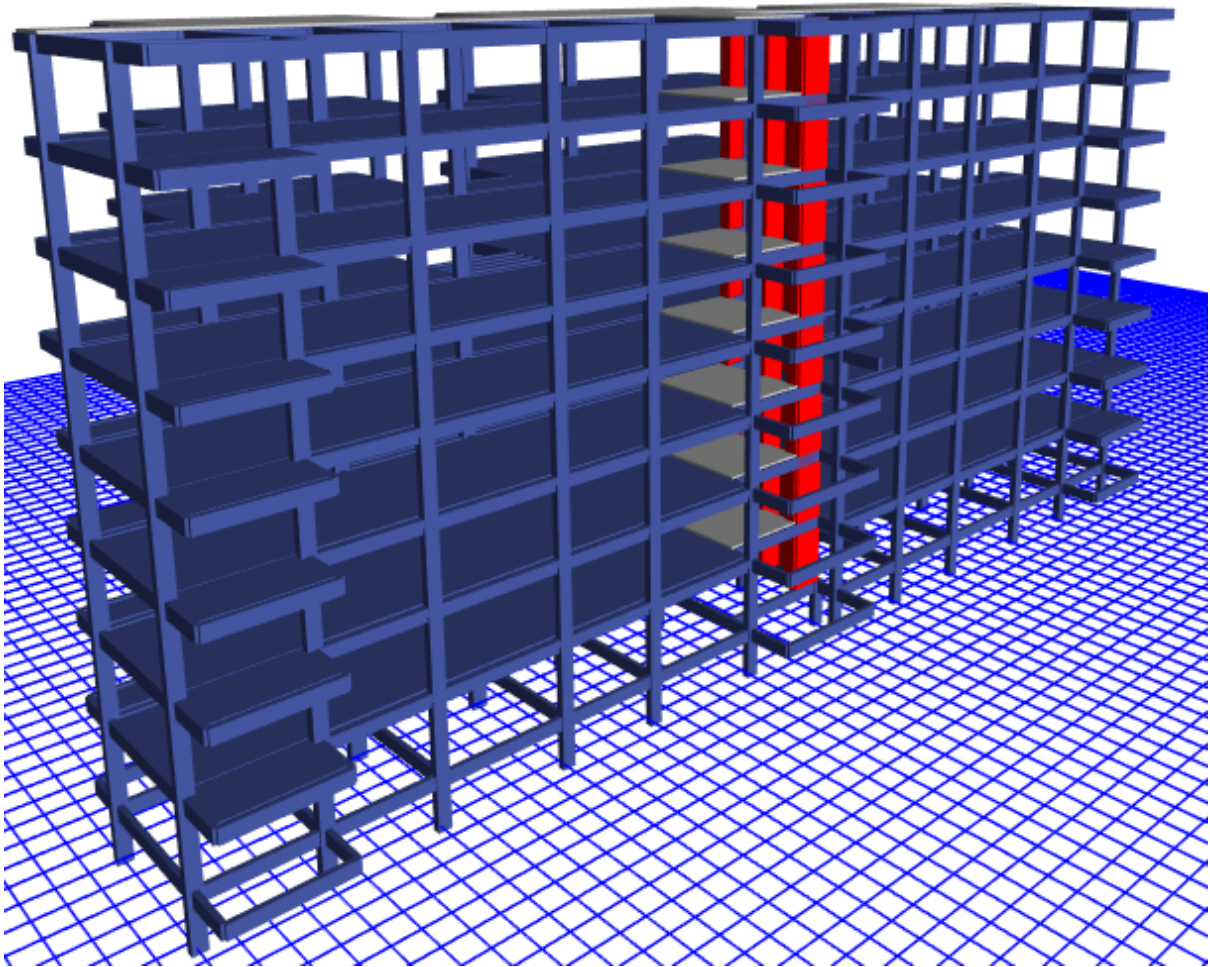


Figure 4.81. Three dimensional Eight Storey RC Building with Shear Wall

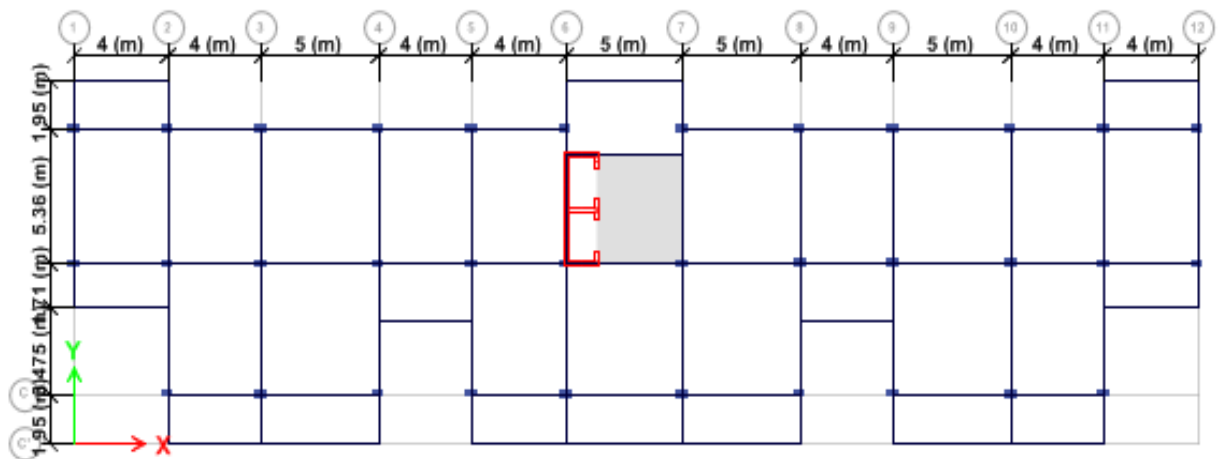


Figure 4.82 Typical Plan View of Eight Storey RC building with Shear Wall

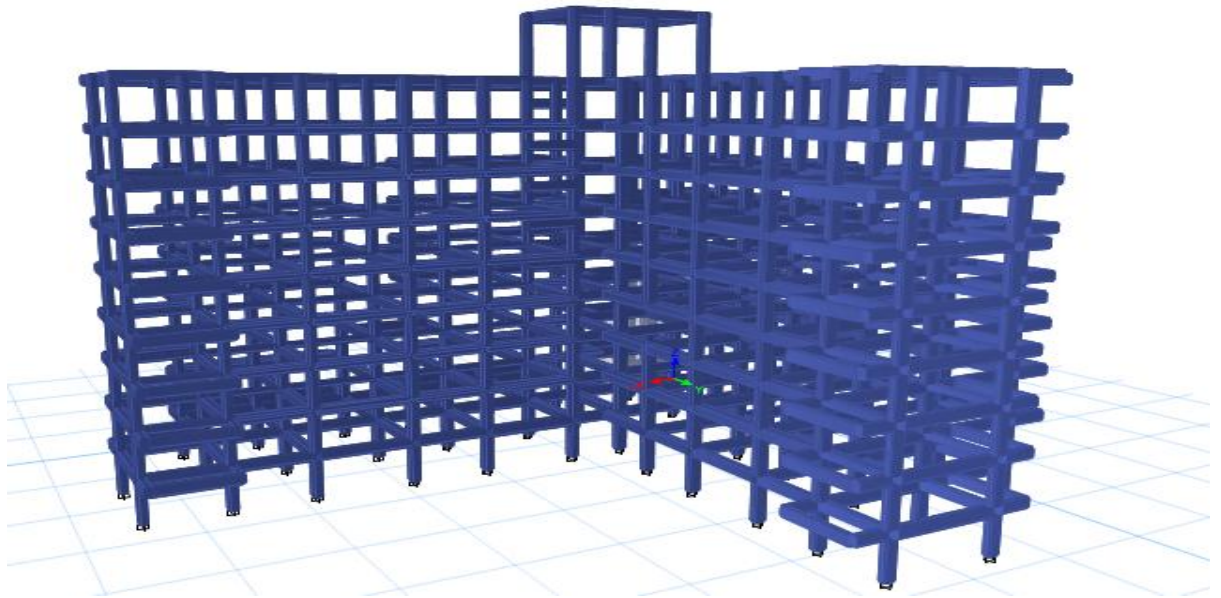


Figure 4.91. Three dimensional Eight Storey Nonlinear or RC L-Shaped Building without shear wall

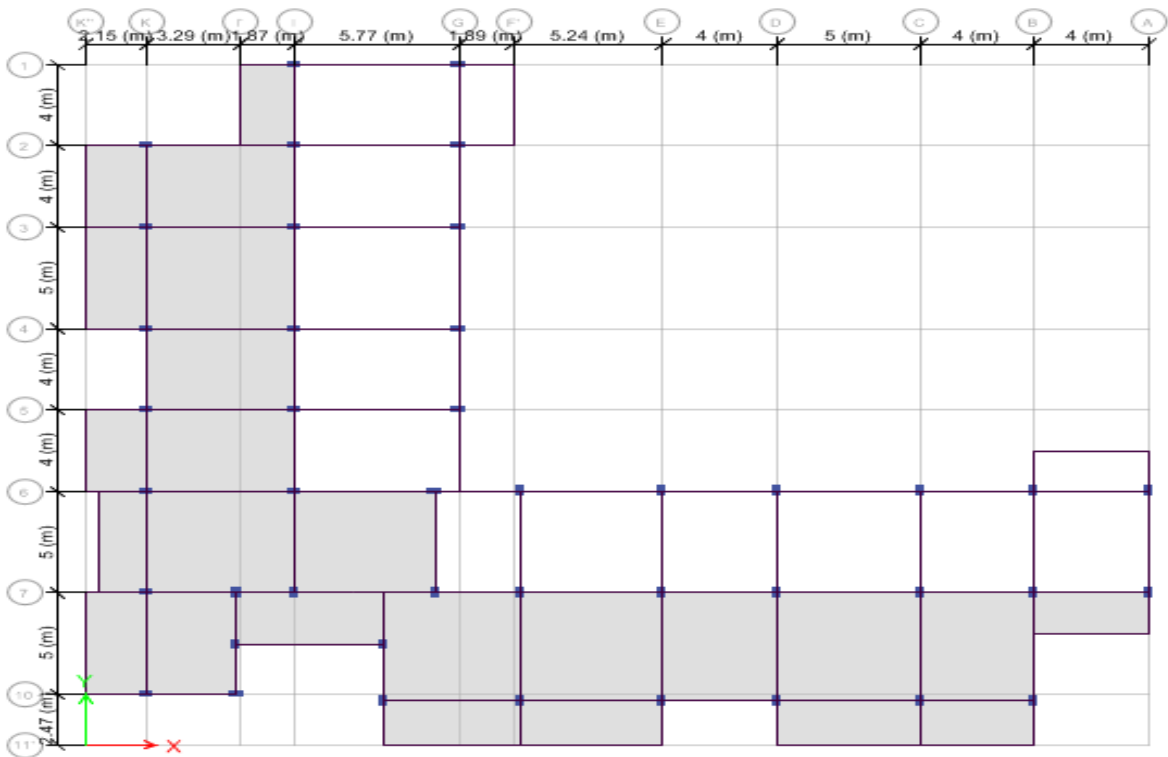


Figure 4.92. Typical Plan view Eight Storey Nonlinear or RC L-Shaped Building without shear wall

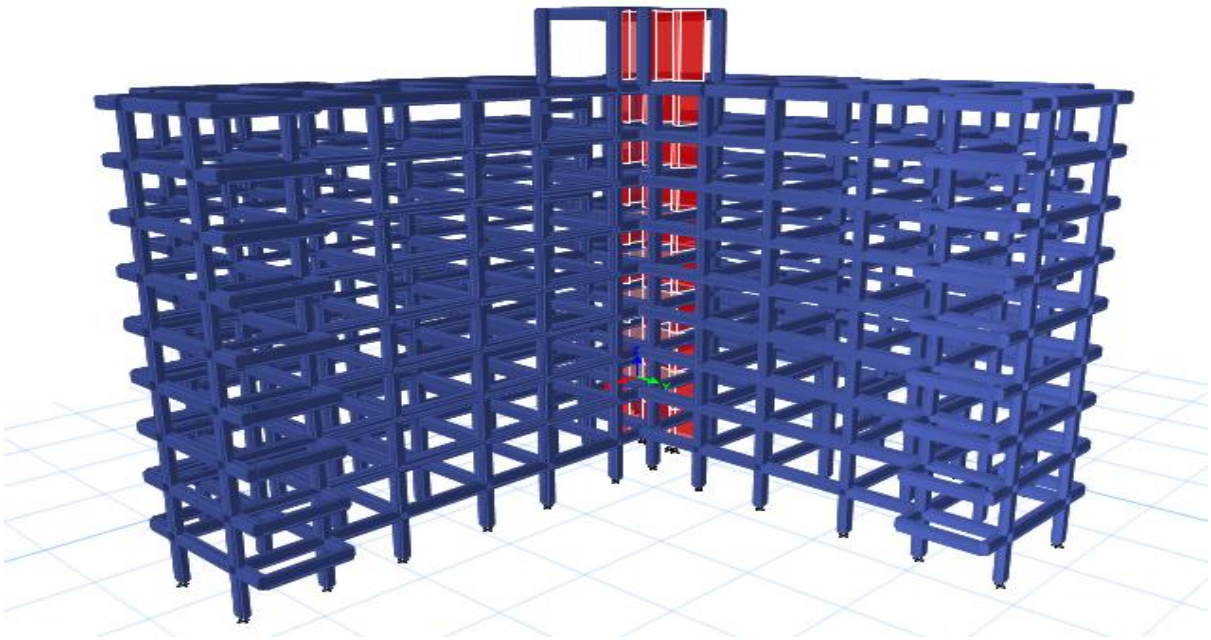


Figure 4.93. Three dimensional Eight Storey Nonlinear or RC L-Shaped Building with shear wall

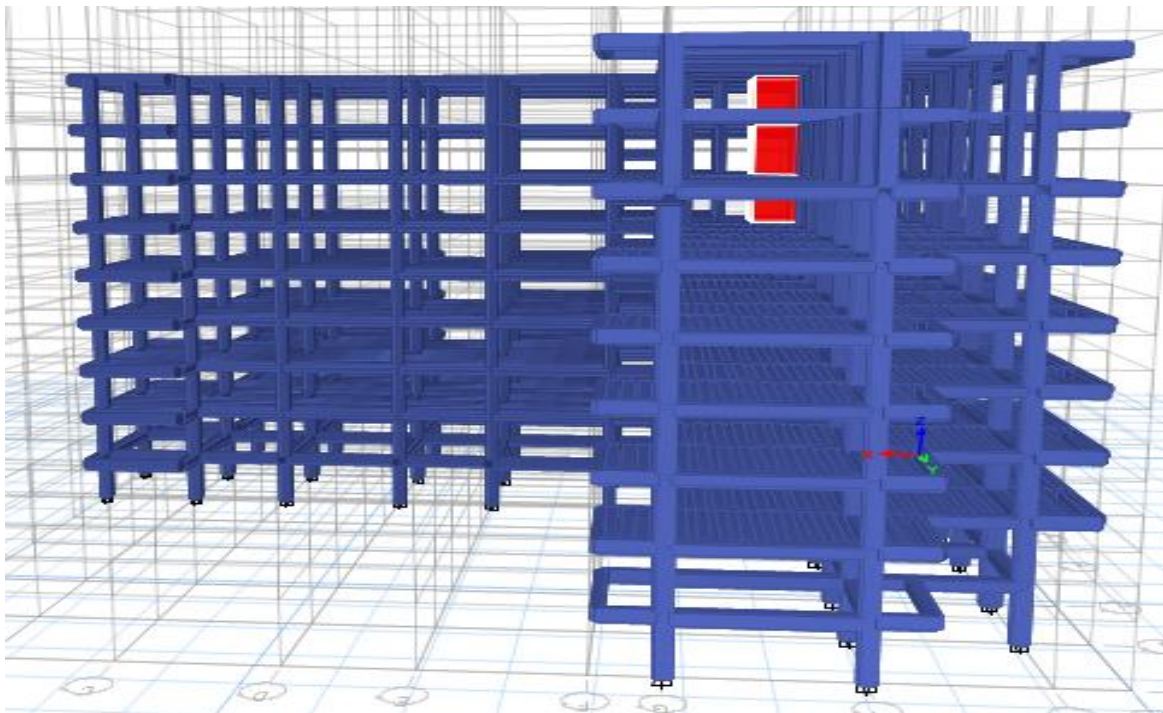


Figure 4.94. Three dimensional Eight Storey Nonlinear or RC L-Shaped Building with shear wall

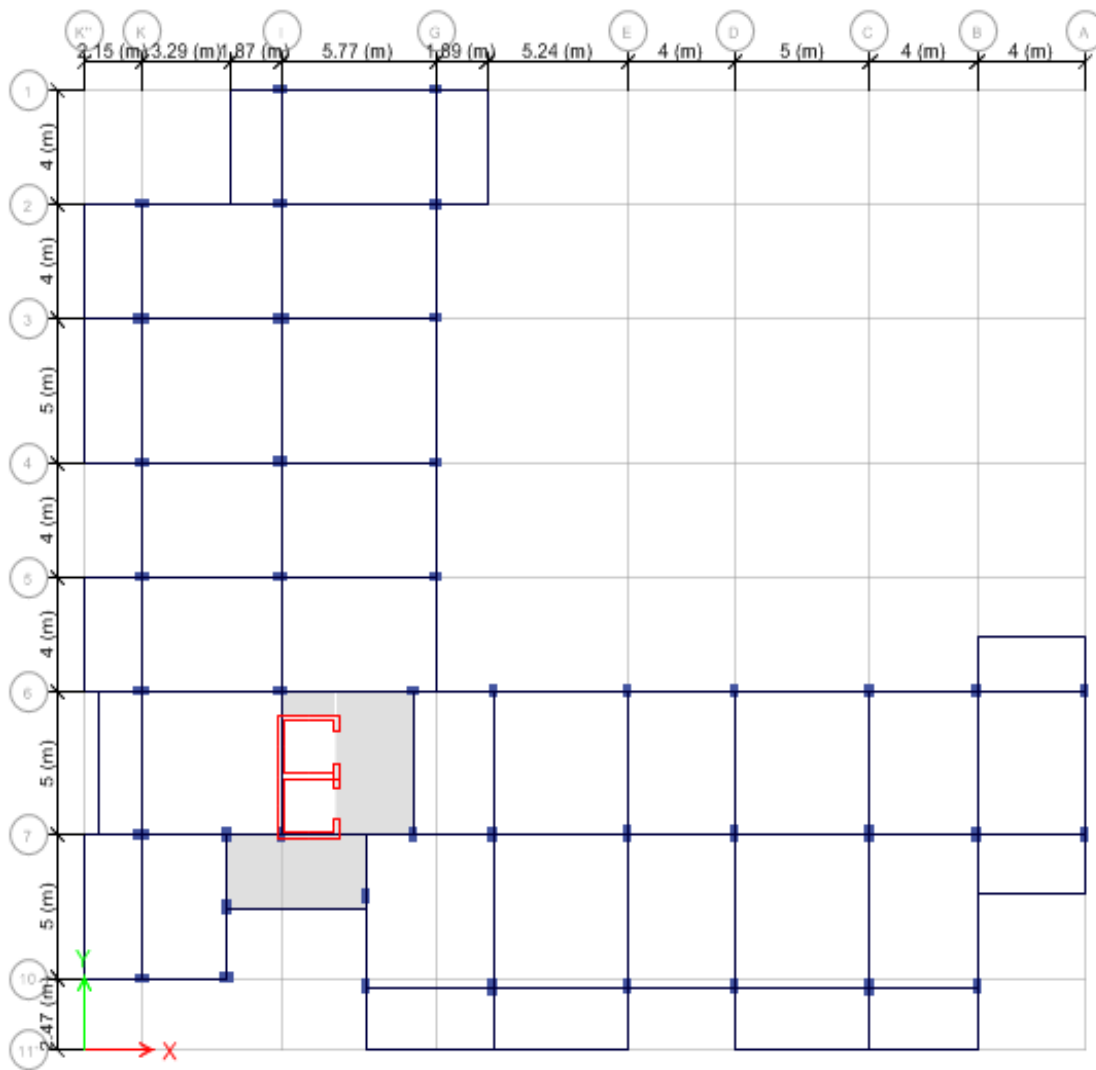


Figure 4.95. Three dimensional and Plan view Eight Storey RC L-Shaped or Nonlinear Building with Shear Wall

### 4.3.1 Columns and Beams Cross Sectional Dimensions and Reinforcements for Linear RC

Building without Shear Wall

#### I. For Eight Storey Rectangular Building

Table 4.1 Dimensions and reinforcements used in eight storey RC building modal for beams

Section	Beam Dimension		Beam and Rein.		Transverse Rein	Remark
	bw(cm)	h(cm)	Top	Bottom		
GFB	20	50	2 $\phi$ 16	2 $\phi$ 16	$\Phi$ 8c/c 200	Ground Floor Beam
FFB Section1-1	20	50	2 $\phi$ 16	2 $\phi$ 14	$\Phi$ 8c/c 200	Frist Floor Beam
FFB Section2-2	20	50	2 $\phi$ 16	2 $\phi$ 16	$\Phi$ 8c/c 200	
FFB Section3-3	40	28	4 $\phi$ 14	4 $\phi$ 14	$\Phi$ 10c/c 200	
FFB Section4-4	46	28	4 $\phi$ 14	4 $\phi$ 14	$\Phi$ 8c/c 200	
FFB Section5-5	20	28	5 $\phi$ 16	5 $\phi$ 16	$\Phi$ 8c/c 200	
FFB Section6-6	20	50	6 $\phi$ 20	4 $\phi$ 20	$\Phi$ 8c/c 200	
FFB Section7-7	20	50	4 $\phi$ 16	2 $\phi$ 14	$\Phi$ 10c/c 200	
FFB Section8-8	20	50	4 $\phi$ 20	4 $\phi$ 16	$\Phi$ 8c/c 200	
FFB Section9-9	20	50	6 $\phi$ 20	5 $\phi$ 16	$\Phi$ 8c/c 200	
FFB Section10-10	20	50	5 $\phi$ 16	3 $\phi$ 14	$\Phi$ 8c/c 200	
FFB Section11-11	20	50	5 $\phi$ 20	5 $\phi$ 16	$\Phi$ 8c/c 200	
FFB Section12-12	20	50	5 $\phi$ 16	3 $\phi$ 16	$\Phi$ 8c/c 200	
FFB Section13-13	20	50	4 $\phi$ 20	2 $\phi$ 16	$\Phi$ 10c/c 200	
FB(2 <sup>th</sup> -7 <sup>th</sup> ) Section1-1	20	50	2 $\phi$ 16	2 $\phi$ 14	$\Phi$ 8- $\Phi$ 10c/c 200	Floor Beam
FB(2 <sup>th</sup> -7 <sup>th</sup> ) Section2-2	20	50	2 $\phi$ 16	2 $\phi$ 16	$\Phi$ 8- $\Phi$ 10c/c 200	
FB(2 <sup>th</sup> -7 <sup>th</sup> ) Section3-3	40	28	4 $\phi$ 14	4 $\phi$ 14	$\Phi$ 8- $\Phi$ 10c/c 200	
FB(2 <sup>th</sup> -7 <sup>th</sup> ) Section4-4	46	28	4 $\phi$ 14	4 $\phi$ 14	$\Phi$ 8- $\Phi$ 10c/c 200	
FB(2 <sup>th</sup> -7 <sup>th</sup> ) Section5-5	40	28	5 $\phi$ 16	5 $\phi$ 16	$\Phi$ 8- $\Phi$ 10c/c 200	
FB(2 <sup>th</sup> -7 <sup>th</sup> ) Section6-6	20	50	6 $\phi$ 20	4 $\phi$ 20	$\Phi$ 8- $\Phi$ 10c/c 200	
FB(2 <sup>th</sup> -7 <sup>th</sup> ) Section7-7	20	50	4 $\phi$ 16	2 $\phi$ 14	$\Phi$ 8- $\Phi$ 10c/c 200	
FB(2 <sup>th</sup> -7 <sup>th</sup> ) Section8-8	20	50	4 $\phi$ 20	4 $\phi$ 16	$\Phi$ 8- $\Phi$ 10c/c 200	
FB(2 <sup>th</sup> -7 <sup>th</sup> ) Section9-9	20	50	6 $\phi$ 20	5 $\phi$ 16	$\Phi$ 8- $\Phi$ 10c/c 200	
FB(2 <sup>th</sup> -7 <sup>th</sup> ) Section10-10	20	50	5 $\phi$ 16	3 $\phi$ 14	$\Phi$ 8- $\Phi$ 10c/c 200	
FB(2 <sup>th</sup> -7 <sup>th</sup> ) Section11-11	20	50	5 $\phi$ 20	5 $\phi$ 16	$\Phi$ 8- $\Phi$ 10c/c 200	
FB(2 <sup>th</sup> -7 <sup>th</sup> ) Section12-12	20	50	4 $\phi$ 20	3 $\phi$ 16	$\Phi$ 8- $\Phi$ 10c/c 200	
FB(2 <sup>th</sup> -7 <sup>th</sup> ) Section13-13	20	50	4 $\phi$ 33	2 $\phi$ 16	$\Phi$ 8- $\Phi$ 10c/c 200	
Rib Section 1-1	12	26	1 $\Phi$ 10	2 $\phi$ 12	$\Phi$ 6c/c 500	Ribbed slab
Rib Section 2-2	12	26	1 $\Phi$ 10	3 $\phi$ 12	$\Phi$ 6c/c 500	
Rib Section 3-3	12	26	1 $\Phi$ 10	2 $\phi$ 10	$\Phi$ 6c/c 500	
Rib Section 4-4	12	26	1 $\Phi$ 10	3 $\phi$ 10	$\Phi$ 6c/c 500	

Table 4.2 Dimensions and reinforcements used in eight storey Rectangular RC building modal for column

Section Name	Column Dimension		Column Longitudenal	Transverse Reinforcement
	bw(cm)	h(cm)	Reinforcement	
CL Section 1-1	50	30	12 $\phi$ 16	$\phi$ 8 clc 190
CL Section 2-2	50	30	8 $\phi$ 16	$\phi$ 8 clc 190
CL Section 3-3	50	30	8 $\phi$ 16	$\phi$ 8 clc 160
CL Section 4-4	60	35	12 $\phi$ 20	$\phi$ 8 clc 200
CL Section 5-5	55	35	12 $\phi$ 16	$\phi$ 8 clc 190
CL Section 6-6	55	35	8 $\phi$ 16	$\phi$ 8 clc 190
CL Section 7-7	50	35	12 $\phi$ 20	$\phi$ 8 clc 200
CL Section 8-8	50	35	12 $\phi$ 16	$\phi$ 8 clc 190
CL Section 9-9	50	30	12 $\phi$ 20	$\phi$ 8 clc 200
CL Section 10-10	50	30	12 $\phi$ 14	$\phi$ 8 clc 160

#### 4.3.2 Columns and Beams Cross Sectional Dimensions and Reinforcements for Linear RC

Building with Shear Wall

Table 4.3 Dimensions and reinforcements used in eight storey RC building models for

Columns

Section Name	Column Dimension		Column Longitudenal	Transverse Reinforcement
	bw(cm)	h(cm)	Reinforcement	
CL Section 1-1	50	30	12 $\phi$ 16	$\phi$ 8 clc 190
CL Section 2-2	50	30	8 $\phi$ 16	$\phi$ 8 clc 190
CL Section 3-3	50	30	8 $\phi$ 16	$\phi$ 8 clc 160
CL Section 4-4	60	35	12 $\phi$ 20	$\phi$ 8 clc 200
CL Section 5-5	55	35	12 $\phi$ 16	$\phi$ 8 clc 190
CL Section 6-6	55	35	8 $\phi$ 16	$\phi$ 8 clc 190
CL Section 7-7	50	35	12 $\phi$ 20	$\phi$ 8 clc 200
CL Section 8-8	50	35	12 $\phi$ 16	$\phi$ 8 clc 190
CL Section 9-9	50	30	12 $\phi$ 20	$\phi$ 8 clc 200
CL Section 10-10	50	30	12 $\phi$ 14	$\phi$ 8 clc 160

Table 4.4 Dimensions and reinforcement used in eight storey RC building models for beams

Section	Beam Dimension		Beam and Rein.		Transverse Rein.	Remark
	bw(cm)	h(cm)	Top	Bottom		
GFB	20	50	2 $\phi$ 16	2 $\phi$ 16	$\Phi$ 8c/c 200	Ground Floor Beam
FFB Section1-1	20	50	2 $\phi$ 16	2 $\phi$ 14	$\Phi$ 8c/c 200	Frist Floor Beam
FFB Section2-2	20	50	2 $\phi$ 16	2 $\phi$ 16	$\Phi$ 8c/c 200	
FFB Section3-3	40	28	4 $\phi$ 14	4 $\phi$ 14	$\Phi$ 10c/c 200	
FFB Section4-4	46	28	4 $\phi$ 14	4 $\phi$ 14	$\Phi$ 8c/c 200	
FFB Section5-5	20	28	5 $\phi$ 16	5 $\phi$ 16	$\Phi$ 8c/c 200	
FFB Section6-6	20	50	6 $\phi$ 20	4 $\phi$ 20	$\Phi$ 8c/c 200	
FFB Section7-7	20	50	4 $\phi$ 16	2 $\phi$ 14	$\Phi$ 10c/c 200	
FFB Section8-8	20	50	4 $\phi$ 20	4 $\phi$ 16	$\Phi$ 8c/c 200	
FFB Section9-9	20	50	6 $\phi$ 20	5 $\phi$ 16	$\Phi$ 8c/c 200	
FFB Section10-10	20	50	5 $\phi$ 16	3 $\phi$ 14	$\Phi$ 8c/c 200	
FFB Section11-11	20	50	5 $\phi$ 20	5 $\phi$ 16	$\Phi$ 8c/c 200	
FFB Section12-12	20	50	5 $\phi$ 16	3 $\phi$ 16	$\Phi$ 8c/c 200	
FFB Section13-13	20	50	4 $\phi$ 20	2 $\phi$ 16	$\Phi$ 10c/c 200	
FB(2 <sup>th</sup> -7 <sup>th</sup> ) Section1-1	20	50	2 $\phi$ 16	2 $\phi$ 14	$\Phi$ 8- $\Phi$ 10c/c 200	Floor Beam
FB(2 <sup>th</sup> -7 <sup>th</sup> ) Section2-2	20	50	2 $\phi$ 16	2 $\phi$ 16	$\Phi$ 8- $\Phi$ 10c/c 200	
FB(2 <sup>th</sup> -7 <sup>th</sup> ) Section3-3	40	28	4 $\phi$ 14	4 $\phi$ 14	$\Phi$ 8- $\Phi$ 10c/c 200	
FB(2 <sup>th</sup> -7 <sup>th</sup> ) Section4-4	46	28	4 $\phi$ 14	4 $\phi$ 14	$\Phi$ 8- $\Phi$ 10c/c 200	
FB(2 <sup>th</sup> -7 <sup>th</sup> ) Section5-5	40	28	5 $\phi$ 16	5 $\phi$ 16	$\Phi$ 8- $\Phi$ 10c/c 200	
FB(2 <sup>th</sup> -7 <sup>th</sup> ) Section6-6	20	50	6 $\phi$ 20	4 $\phi$ 20	$\Phi$ 8- $\Phi$ 10c/c 200	
FB(2 <sup>th</sup> -7 <sup>th</sup> ) Section7-7	20	50	4 $\phi$ 16	2 $\phi$ 14	$\Phi$ 8- $\Phi$ 10c/c 200	
FB(2 <sup>th</sup> -7 <sup>th</sup> ) Section8-8	20	50	4 $\phi$ 20	4 $\phi$ 16	$\Phi$ 8- $\Phi$ 10c/c 200	
FB(2 <sup>th</sup> -7 <sup>th</sup> ) Section9-9	20	50	6 $\phi$ 20	5 $\phi$ 16	$\Phi$ 8- $\Phi$ 10c/c 200	
FB(2 <sup>th</sup> -7 <sup>th</sup> ) Section10-10	20	50	5 $\phi$ 16	3 $\phi$ 14	$\Phi$ 8- $\Phi$ 10c/c 200	
FB(2 <sup>th</sup> -7 <sup>th</sup> ) Section11-11	20	50	5 $\phi$ 20	5 $\phi$ 16	$\Phi$ 8- $\Phi$ 10c/c 200	
FB(2 <sup>th</sup> -7 <sup>th</sup> ) Section12-12	20	50	4 $\phi$ 20	3 $\phi$ 16	$\Phi$ 8- $\Phi$ 10c/c 200	
FB(2 <sup>th</sup> -7 <sup>th</sup> ) Section13-13	20	50	4 $\phi$ 33	2 $\phi$ 16	$\Phi$ 8- $\Phi$ 10c/c 200	
Rib Section 1-1	15	20	1 $\Phi$ 10	2 $\phi$ 12	$\Phi$ 6c/c 500	Ribbed slab
Rib Section 2-2	15	30	1 $\Phi$ 10	3 $\phi$ 12	$\Phi$ 6c/c 500	
Rib Section 3-3	15	20	1 $\Phi$ 10	2 $\phi$ 10	$\Phi$ 6c/c 500	
Rib Section 4-4	15	30	1 $\Phi$ 10	3 $\phi$ 10	$\Phi$ 6c/c 500	
Shear Wall	230	450	8 $\Phi$ 16	8 $\Phi$ 16	$\Phi$ 10c/c 200	
			4 $\Phi$ 16	4 $\Phi$ 16	$\Phi$ 10c/c 200	
					$\Phi$ 8 Horizontal c/c 500	
					$\Phi$ 8 Vertical c/c 600	

### 4.3.3 Columns and Beams Cross Sectional Dimensions and Reinforcements for L-Shaped RC

Building or Non linear without Shear Wall

Table 4.5 Dimensions and reinforcements used in eight storeys RC building model for beams

	bw(cm)	h(cm)	Top	Bottom		
GFB	20	50	2 $\phi$ 16	2 $\phi$ 16	$\Phi$ 8-10 $\phi$ c/c 200	Ground Floor Beam
FFB Section1-1	20	50	2 $\phi$ 16	2 $\phi$ 16	$\Phi$ 8-10 $\phi$ c/c 200	Frist Floor Beam
FFB Section2-2	20	50	3 $\phi$ 16	2 $\phi$ 14	$\Phi$ 8-10 $\phi$ c/c 200	
FFB Section3-3	20	50	2 $\phi$ 16	2 $\phi$ 14	$\Phi$ 8-10 $\phi$ c/c 200	
FFB Section4-4	20	50	5 $\phi$ 20	3 $\phi$ 14	$\Phi$ 8-10 $\phi$ c/c 200	
FFB Section5-5	20	50	5 $\phi$ 20	3 $\phi$ 14	$\Phi$ 8-10 $\phi$ c/c 200	
FFB Section6-6	20	50	5 $\phi$ 16	3 $\phi$ 14	$\Phi$ 8-10 $\phi$ c/c 200	
FFB Section7-7	20	50	7 $\phi$ 20	4 $\phi$ 20	$\Phi$ 8-10 $\phi$ c/c 200	
FFB Section8-8	40	30	4 $\phi$ 16	4 $\phi$ 14	$\Phi$ 8-10 $\phi$ c/c 200	
FFB Section9-9	20	50	6 $\phi$ 20	5 $\phi$ 16	$\Phi$ 8-10 $\phi$ c/c 200	
FFB Section10-10	20	50	5 $\phi$ 20	5 $\phi$ 16	$\Phi$ 8-10 $\phi$ c/c 200	
FFB Section11-11	20	30	4 $\phi$ 14	4 $\phi$ 16	$\Phi$ 8-10 $\phi$ c/c 200	
FFB Section12-12	20	50	5 $\phi$ 20	4 $\phi$ 14	$\Phi$ 8-10 $\phi$ c/c 200	
FFB Section13-13	20	50	4 $\phi$ 20	5 $\phi$ 16	$\Phi$ 8-10 $\phi$ c/c 200	
FFB Section14-14	20	50	4 $\phi$ 20	4 $\phi$ 16	$\Phi$ 8-10 $\phi$ c/c 200	
FFB Section15-15	20	50	3 $\phi$ 20	2 $\phi$ 16	$\Phi$ 8-10 $\phi$ c/c 200	
FFB Section16-16	20	50	3 $\phi$ 14	3 $\phi$ 14	$\Phi$ 8-10 $\phi$ c/c 200	
FB(2 <sup>th</sup> -7 <sup>th</sup> ) Section1-1	20	50	2 $\phi$ 16	2 $\phi$ 14	$\Phi$ 8- $\Phi$ 10c/c 200	Floor Beam
FB(2 <sup>th</sup> -7 <sup>th</sup> ) Section2-2	40	30	4 $\phi$ 14	4 $\phi$ 14	$\Phi$ 8- $\Phi$ 10c/c 200	
FB(2 <sup>th</sup> -7 <sup>th</sup> ) Section3-3	20	50	5 $\phi$ 16	3 $\phi$ 14	$\Phi$ 8- $\Phi$ 10c/c 200	
FB(2 <sup>th</sup> -7 <sup>th</sup> ) Section4-4	20	50	2 $\phi$ 16	2 $\phi$ 16	$\Phi$ 8- $\Phi$ 10c/c 200	
FB(2 <sup>th</sup> -7 <sup>th</sup> ) Section5-5	20	30	4 $\phi$ 14	4 $\phi$ 14	$\Phi$ 8- $\Phi$ 10c/c 200	
FB(2 <sup>th</sup> -7 <sup>th</sup> ) Section6-6	20	50	4 $\phi$ 20	4 $\phi$ 20	$\Phi$ 8- $\Phi$ 10c/c 200	
FB(2 <sup>th</sup> -7 <sup>th</sup> ) Section7-7	20	50	4 $\phi$ 20	5 $\phi$ 16	$\Phi$ 8- $\Phi$ 10c/c 200	
FB(2 <sup>th</sup> -7 <sup>th</sup> ) Section8-8	20	50	5 $\phi$ 20	5 $\phi$ 16	$\Phi$ 8- $\Phi$ 10c/c 200	
FB(2 <sup>th</sup> -7 <sup>th</sup> ) Section9-9	20	50	4 $\phi$ 16	4 $\phi$ 14	$\Phi$ 8- $\Phi$ 10c/c 200	
FB(2 <sup>th</sup> -7 <sup>th</sup> ) Section10-10	20	50	3 $\phi$ 14	3 $\phi$ 14	$\Phi$ 8- $\Phi$ 10c/c 200	
FB(2 <sup>th</sup> -7 <sup>th</sup> ) Section11-11	20	50	4 $\phi$ 20	4 $\phi$ 16	$\Phi$ 8- $\Phi$ 10c/c 200	
FB(2 <sup>th</sup> -7 <sup>th</sup> ) Section12-12	20	50	3 $\phi$ 20	3 $\phi$ 14	$\Phi$ 8- $\Phi$ 10c/c 200	
FB(2 <sup>th</sup> -7 <sup>th</sup> ) Section13-13	20	50	2 $\phi$ 16	3 $\phi$ 14	$\Phi$ 8- $\Phi$ 10c/c 200	
FB(2 <sup>th</sup> -7 <sup>th</sup> ) Section14-14	20	50	5 $\phi$ 20	5 $\phi$ 16	$\Phi$ 8- $\Phi$ 10c/c 200	
Rib Section 1-1	15	30	1 $\Phi$ 10	2 $\phi$ 12	$\Phi$ 6c/c 500	Ribbed slab beam
Rib Section 2-2	15	30	1 $\Phi$ 10	3 $\phi$ 12	$\Phi$ 6c/c 500	
Rib Section 3-3	15	30	1 $\Phi$ 10	2 $\phi$ 10	$\Phi$ 6c/c 500	
Rib Section 4-4	15	30	1 $\Phi$ 10	3 $\phi$ 10	$\Phi$ 6c/c 500	
TTB Section 1-1	20	50	3 $\Phi$ 16	2 $\phi$ 14	$\Phi$ 8- $\Phi$ 10c/c 200	Top Tie Beam
TTB Section 2-2	20	50	4 $\Phi$ 16	4 $\phi$ 16	$\Phi$ 8- $\Phi$ 10c/c 200	
TTB Section 3-3	20	50	5 $\Phi$ 16	4 $\phi$ 16	$\Phi$ 8- $\Phi$ 10c/c 200	
TTB Section 4-4	20	50	4 $\Phi$ 16	3 $\phi$ 14	$\Phi$ 8- $\Phi$ 10c/c 200	
TTB Section 5-5	20	50	2 $\Phi$ 16	2 $\phi$ 14	$\Phi$ 8- $\Phi$ 10c/c 200	
TTB Section 6-6	20	50	5 $\Phi$ 20	3 $\phi$ 14	$\Phi$ 8- $\Phi$ 10c/c 200	
TTB Section 7-7	20	50	3 $\Phi$ 16	3 $\phi$ 14	$\Phi$ 8- $\Phi$ 10c/c 200	
TTB Section 8-8	20	50	4 $\Phi$ 16	2 $\phi$ 14	$\Phi$ 8- $\Phi$ 10c/c 200	

Table 4.6 Dimensions and reinforcements used in eight storeys RC building model for columns

Section Name	Column Dimension		Column Longitudenal Reinforcement	Transverse Reinforcement
	bw(cm)	h(cm)		
CL Section 1-1	50	30	12 $\phi$ 16	$\phi$ 8 clc 190
CL Section 2-2	50	30	8 $\phi$ 16	$\phi$ 8 clc 190
CL Section 3-3	50	30	8 $\phi$ 14	$\phi$ 8 clc 160
CL Section 4-4	60	35	12 $\phi$ 20	$\phi$ 8 clc 200
CL Section 5-5	55	35	12 $\phi$ 10	$\phi$ 8 clc 200
CL Section 6-6	55	35	12 $\phi$ 16	$\phi$ 8 clc 190
CL Section 7-7	50	35	12 $\phi$ 20	$\phi$ 8 clc 200
CL Section 8-8	50	35	12 $\phi$ 16	$\phi$ 8 clc 190
CL Section 9-9	50	30	12 $\phi$ 20	$\phi$ 8 clc 200
CL Section 10-10	60	35	16 $\phi$ 20	$\phi$ 8 clc 200
CL Section 11-11	50	35	12 $\phi$ 14	$\phi$ 8 clc 160
CL Section 12-12	50	30	12 $\phi$ 14	$\phi$ 8 clc 160
CL Section 13-13	55	35	16 $\phi$ 20	$\phi$ 8 clc 200
CL Section 14-14	50	30	12 $\phi$ 16	$\phi$ 8 clc 190

#### 4.3.4 Columns and Beams Cross Sectional Dimensions and Reinforcements for L-Shaped RC

Building or Nonlinear with Shear Wall (Appendix 4)

Table 4.7 Dimensions and reinforcements used in eight storey RC building models for Columns

Section Name	Column Dimension		Column Longitudenal Reinforcement	Transverse Reinforcement
	bw(cm)	h(cm)		
CL Section 1-1	50	30	12 $\phi$ 16	$\phi$ 8 clc 190
CL Section 2-2	50	30	8 $\phi$ 16	$\phi$ 8 clc 190
CL Section 3-3	50	30	8 $\phi$ 14	$\phi$ 8 clc 160
CL Section 4-4	60	35	12 $\phi$ 20	$\phi$ 8 clc 200
CL Section 5-5	55	35	12 $\phi$ 10	$\phi$ 8 clc 200
CL Section 6-6	55	35	12 $\phi$ 16	$\phi$ 8 clc 190
CL Section 7-7	50	35	12 $\phi$ 20	$\phi$ 8 clc 200
CL Section 8-8	50	35	12 $\phi$ 16	$\phi$ 8 clc 190
CL Section 9-9	50	30	12 $\phi$ 20	$\phi$ 8 clc 200
CL Section 10-10	60	35	16 $\phi$ 20	$\phi$ 8 clc 200
CL Section 11-11	50	35	12 $\phi$ 14	$\phi$ 8 clc 160
CL Section 12-12	50	30	12 $\phi$ 14	$\phi$ 8 clc 160
CL Section 13-13	55	35	16 $\phi$ 20	$\phi$ 8 clc 200
CL Section 14-14	50	30	12 $\phi$ 16	$\phi$ 8 clc 190

Table 4.8 Dimensions and reinforcements used in eight storey RC building models for beams

Section	Beam Dimension		Beam and Rein.		Transverse Rein.	Remark
	bw(cm)	h(cm)	Top	Bottom		
GFB	20	50	2 $\phi$ 16	2 $\phi$ 16	$\Phi$ 8-10 $\phi$ c/c 200	Ground Floor Beam
FFB Section1-1	20	50	2 $\phi$ 16	2 $\phi$ 16	$\Phi$ 8-10 $\phi$ c/c 200	Frist Floor Beam
FFB Section2-2	20	50	3 $\phi$ 16	2 $\phi$ 14	$\Phi$ 8-10 $\phi$ c/c 200	
FFB Section3-3	20	50	2 $\phi$ 16	2 $\phi$ 14	$\Phi$ 8-10 $\phi$ c/c 200	
FFB Section4-4	20	50	5 $\phi$ 20	3 $\phi$ 14	$\Phi$ 8-10 $\phi$ c/c 200	
FFB Section5-5	20	50	5 $\phi$ 20	3 $\phi$ 14	$\Phi$ 8-10 $\phi$ c/c 200	
FFB Section6-6	20	50	5 $\phi$ 16	3 $\phi$ 14	$\Phi$ 8-10 $\phi$ c/c 200	
FFB Section7-7	20	50	7 $\phi$ 20	4 $\phi$ 20	$\Phi$ 8-10 $\phi$ c/c 200	
FFB Section8-8	40	30	4 $\phi$ 16	4 $\phi$ 14	$\Phi$ 8-10 $\phi$ c/c 200	
FFB Section9-9	20	50	6 $\phi$ 20	5 $\phi$ 16	$\Phi$ 8-10 $\phi$ c/c 200	
FFB Section10-10	20	50	5 $\phi$ 20	5 $\phi$ 16	$\Phi$ 8-10 $\phi$ c/c 200	
FFB Section11-11	20	30	4 $\phi$ 14	4 $\phi$ 16	$\Phi$ 8-10 $\phi$ c/c 200	
FFB Section12-12	20	50	5 $\phi$ 20	4 $\phi$ 14	$\Phi$ 8-10 $\phi$ c/c 200	
FFB Section13-13	20	50	4 $\phi$ 20	5 $\phi$ 16	$\Phi$ 8-10 $\phi$ c/c 200	
FFB Section14-14	20	50	4 $\phi$ 20	4 $\phi$ 16	$\Phi$ 8-10 $\phi$ c/c 200	
FFB Section15-15	20	50	3 $\phi$ 20	2 $\phi$ 16	$\Phi$ 8-10 $\phi$ c/c 200	
FFB Section16-16	20	50	3 $\phi$ 14	3 $\phi$ 14	$\Phi$ 8-10 $\phi$ c/c 200	
FB(2 <sup>th</sup> -7 <sup>th</sup> ) Section1-1	20	50	2 $\phi$ 16	2 $\phi$ 14	$\Phi$ 8- $\Phi$ 10c/c 200	Floor Beam
FB(2 <sup>th</sup> -7 <sup>th</sup> ) Section2-2	40	30	4 $\phi$ 14	4 $\phi$ 14	$\Phi$ 8- $\Phi$ 10c/c 200	
FB(2 <sup>th</sup> -7 <sup>th</sup> ) Section3-3	20	50	5 $\phi$ 16	3 $\phi$ 14	$\Phi$ 8- $\Phi$ 10c/c 200	
FB(2 <sup>th</sup> -7 <sup>th</sup> ) Section4-4	20	50	2 $\phi$ 16	2 $\phi$ 16	$\Phi$ 8- $\Phi$ 10c/c 200	
FB(2 <sup>th</sup> -7 <sup>th</sup> ) Section5-5	20	30	4 $\phi$ 14	4 $\phi$ 14	$\Phi$ 8- $\Phi$ 10c/c 200	
FB(2 <sup>th</sup> -7 <sup>th</sup> ) Section6-6	20	50	4 $\phi$ 20	4 $\phi$ 20	$\Phi$ 8- $\Phi$ 10c/c 200	
FB(2 <sup>th</sup> -7 <sup>th</sup> ) Section7-7	20	50	4 $\phi$ 20	5 $\phi$ 16	$\Phi$ 8- $\Phi$ 10c/c 200	
FB(2 <sup>th</sup> -7 <sup>th</sup> ) Section8-8	20	50	5 $\phi$ 20	5 $\phi$ 16	$\Phi$ 8- $\Phi$ 10c/c 200	
FB(2 <sup>th</sup> -7 <sup>th</sup> ) Section9-9	20	50	4 $\phi$ 16	4 $\phi$ 14	$\Phi$ 8- $\Phi$ 10c/c 200	
FB(2 <sup>th</sup> -7 <sup>th</sup> ) Section10-10	20	50	3 $\phi$ 14	3 $\phi$ 14	$\Phi$ 8- $\Phi$ 10c/c 200	
FB(2 <sup>th</sup> -7 <sup>th</sup> ) Section11-11	20	50	4 $\phi$ 20	4 $\phi$ 16	$\Phi$ 8- $\Phi$ 10c/c 200	
FB(2 <sup>th</sup> -7 <sup>th</sup> ) Section12-12	20	50	3 $\phi$ 20	3 $\phi$ 14	$\Phi$ 8- $\Phi$ 10c/c 200	
FB(2 <sup>th</sup> -7 <sup>th</sup> ) Section13-13	20	50	2 $\phi$ 16	3 $\phi$ 14	$\Phi$ 8- $\Phi$ 10c/c 200	
FB(2 <sup>th</sup> -7 <sup>th</sup> ) Section14-14	20	50	5 $\phi$ 20	5 $\phi$ 16	$\Phi$ 8- $\Phi$ 10c/c 200	
Rib Section 1-1	15	30	1 $\Phi$ 10	2 $\phi$ 12	$\Phi$ 6c/c 500	Ribbed slab beam
Rib Section 2-2	15	30	1 $\Phi$ 10	3 $\phi$ 12	$\Phi$ 6c/c 500	
Rib Section 3-3	15	30	1 $\Phi$ 10	2 $\phi$ 10	$\Phi$ 6c/c 500	
Rib Section 4-4	15	30	1 $\Phi$ 10	3 $\phi$ 10	$\Phi$ 6c/c 500	
TTB Section 1-1	20	50	3 $\Phi$ 16	2 $\phi$ 14	$\Phi$ 8- $\Phi$ 10c/c 200	Top Tie Beam
TTB Section 2-2	20	50	4 $\Phi$ 16	4 $\phi$ 16	$\Phi$ 8- $\Phi$ 10c/c 200	
TTB Section 3-3	20	50	5 $\Phi$ 16	4 $\phi$ 16	$\Phi$ 8- $\Phi$ 10c/c 200	
TTB Section 4-4	20	50	4 $\Phi$ 16	3 $\phi$ 14	$\Phi$ 8- $\Phi$ 10c/c 200	
TTB Section 5-5	20	50	2 $\Phi$ 16	2 $\phi$ 14	$\Phi$ 8- $\Phi$ 10c/c 200	
TTB Section 6-6	20	50	5 $\Phi$ 20	3 $\phi$ 14	$\Phi$ 8- $\Phi$ 10c/c 200	
TTB Section 7-7	20	50	3 $\Phi$ 16	3 $\phi$ 14	$\Phi$ 8- $\Phi$ 10c/c 200	

## **CHAPTER FIVE-PUSHOVER ANALYSIS OF BUILDINGS AND COMPARISONS**

The pushover analysis can be used to evaluate the expected performance of a structure system by estimating its strength and deformation demands for design earthquakes by means of static inelastic analysis, and comparing these demands with available capacities at the performance level of interest. It refers to an analysis procedure where by an incremental-iterative solution of the static equilibrium equations has been carried out to obtain the response of a structure subjected to monotonically increasing lateral pattern. The structural resistance is evaluated and the stiffness matrix is updated at each increment of the forcing function, up to convergence. The solution proceeds until (i) a predefined performance limit states is reached, (ii) structural collapse is observed or (iii) the program fails to converge. In this manner, each point in the resulting displacement vs. base shear capacity curve represents effective and equilibrated stress states of the structure. i.e., a state of deformation that bears a direct correspondence to the applied external force vector, [2].

The NSPs are generally believed to be superior than Linear Elastic Procedures (LSPs), such as the classic equivalent static lateral force procedures and modal superposition techniques, because they explicitly consider inelasticity of yielding-expected structural components in resisting moderate and large earthquake intensities. Furthermore, the NSPs are more appealing than Nonlinear Dynamic Procedures (NDPs), which are considered to be the most sophisticated of all available seismic analysis methods, as they yield single-valued estimates of response quantities (e.g., lateral displacement , interstory drifts, member forces and moments, and plastic hinge rotations) for design or evaluation. In pushover analysis, the seismic demands are estimated by the nonlinear static analysis of structure subjected to monotonically increasing lateral forces varying through the height of the structure. The analysis is carried out by applying the gravity loads following by lateral loading along a direction starting at the end of the gravity push. The structure is pushed until either a predetermined target displacement is reached or it collapse. The reliable post-yield material model and inelastic member deformations are extremely important parameters, including global drift, inter-storey drift, inelastic element deformations (either absolute or normalized with respect to a yield valve), deformations between elements, and element and connection forces( for elements and connections that cannot sustain inelastic deformation). The inelastic static pushover analysis can be viewed as a method for predicting seismic force and deformation demands, which accounts in an approximate manner for the redistribution of internal forces occurring when the structure is subjected to inertia forces that no longer can be

resisted within the elastic range of structural behaviour. The two key steps in applying this method, i.e. lateral force distribution and target displacement are based on the assumption that the structures response is mainly from the fundamental mode, and that the mode shapes remain unchanged after structure gets into the inelastic region,[2].

### **5.1. Statics Nonlinear Pushover Analysis**

Nonlinear static pushover analysis has become the most commonly used methods determine the nonlinear behaviour of the building structures in the recent years. In this simplified method, a capacity curve is obtained which shows the relation of base shear and roof displacement. This curve represents the behaviour of the building structure under increasing base shear forces. As the capacities of the members of the lateral force resisting system exceed their yield limits during the increasing of the base shear forces, the slope of the force-deformation curve will change, and hence the nonlinear behaviour can be represented. In the pushover analysis, the applied lateral forces are calculated for each step and the stiffness of the members whose capacities are exceeded is changed according to the hinge properties in the next step of the analysis. This process ends when the structure becomes unstable.

The pushover analysis can be performed considering the control over the force or displacement. Force control option is useful when the magnitude of the load is known clearly, and the structure is expected to support that load. The displacements are searched.

In this study, due to its simplicity and its computation power SAP2000 and ETABS computer software in utilized to carry out the pushover analyses, [2].

#### **5.1.1 Purpose of Non-linear Static Pushover Analysis**

The purpose of pushover analysis is to evaluate the expected the expected performance of structural systems by estimating performance of a structure system by estimating its strength and deformation demands in design earthquakes by means of static inelastic analysis, and comparing these demands to available capacities at the performance levels of interest. The evaluation is based on an assessment of important performance parameters, including global drift, interstory drift, inelastic element deformation ( either absolute or normalized with respect to a yield value), deformations between elements, and element connection forces( for elements and connections that cannot sustain inelastic deformations), The inelastic static pushover analysis can be viewed as a method for predicting seismic force and deformation demands, which accounts in an approximate manner for the redistribution of internal forces

that no longer can be resisted within the elastic range of structural behaviour. The pushover is expected to provide information on many response characteristics that can be obtained from an elastic static or dynamic analysis.

The following are the examples of such response characteristics

- The realistic force demands on brace connections, moment demands on beam to columns, force demands on brace connections, moment demands on beam to column connections, shear force demands in deep reinforced concrete spandrel beams, shear force demands in unreinforced masonry wall piers, etc.
- Estimates of the deformations demands for elements that have to form in elastically in order to dissipate the energy imparted to the structure.
- Consequences of the strength deterioration of individual elements on behaviour of structural system.
- Identification of the critical regions in which the deformation demands are expected to be high and that have to become the focus through detailing.
- Identification of the strength discontinuities in plan elevation that will lead to changes in the dynamic characteristic in elastic range.
- Estimates of the interstory drifts that account for strength or stiffness discontinuities and that may be used to control the damages and to evaluate P-Delta effects.
- Verification of the completeness and adequacy of load path, considering all the elements of the structural system, all the connections, the stiff non-structural elements of significant strength, and foundation system.

The last item is most relevant one as the analytical model incorporates all elements, whether structural or non structural, that contribute significantly to the lateral load distribution. Load transfer through across the connections through the ductile elements can be checked with realistic forces; and the maximum overturning moment in walls, which is often limited by the uplift capacity of foundation elements can be estimated, [2].

## 5.2 Steps in pushover Analysis

The following steps are included in the pushover analysis. Step 1 through 4 discuss creating the computer model, Steps 5 runs the analysis, and Steps 6 through 10 review the pushover analysis results.

- Create the basic computer model( without the pushover data) in the usual manner using the graphical interface of SAP2000/ETABS makes this a quick and easy task.
- Define properties and acceptance criteria for pushover hinges.
- The program includes several built-in default hinge properties that are based on average values from ATC-40 for concrete members and average values from FEMA-273 for steel members.
- Locate the pushover hinges on the model by selecting one or more frame members and assigning them one or more hinge properties and hinge locations considering end-offsets.
- Define the pushover load cases. In SAP2000/ETABS more than one pushover load can be run in the same analysis. Also a pushover load case start from the final conditions of another pushover load case that was previously run the same analysis.

Typically the first pushover load case is used to apply gravity load and then subsequent lateral pushover load cases are specified to start from the final condition of the gravity pushover. Pushover load cases can be force controlled, that is, pushed to a specified displacement. Typically a gravity load pushover is force controlled and lateral pushovers are displacement controlled

- Run the basic static analysis and, if desired, dynamic analysis. Then run the static nonlinear pushover analysis.
- Display the pushover curve.
- Display the capacity spectrum curve.

We can interactively modify the magnitude of the earthquake and the damping information on this form and immediately see the new capacity spectrum plot.

The performance point for a given set of values is defined by the intersection of the capacity curve and the single demand spectrum curve. Also, the file menu in this display allows you to print the coordinates of the capacity curve and the demand curve as well as other information used to convert the pushover curve to Acceleration-Displacement Response Spectrum (ADRS) format.

- Review the pushover displaced shape and sequence of hinge formation on a step-by-step basis.
- Review member forces on a step-by-step basis.
- Output for the pushover analysis can be printed in a tabular form for the entire model or for selected elements of the model. The types of output available in this form include joint displacements at each step of the pushover, frame member forces at each step of the pushover, and hinge force, displacement and state at each step of the pushover,[9] and[13].

### **5.3 Effective Stiffness Factors**

It is necessary to define effective elastic properties to define the elastic part of the lumped-plasticity model since the elastic part of the member cracks when the member enters in to inelastic range. Hence, the stiffness reduction factors were used to reduce the gross stiffness properties of the beams and the columns and the reduced stiffness were used to define the effective stiffness model and the lumped-plasticity model. The effective rigidity assumed for each member of the structure should be consistent with its intended behaviour and the model used. For lumped plasticity elements, it is recommended( Kappos,1986;Paulay and Priestly,1992;Penelis and Kappos, 1997; ASCE,2000) to use 30% to 50% of the gross flexural rigidity( $EI_g$ ) for beams, 40% to 60%  $EI_g$  for the walls, and 60% to 80%  $EI_g$  for columns( in compression), to account for member cracking, which is different in each type of member; the higher values for columns and walls apply for high axial compression. The approximate component initial effective stiffness values according to ATC-40;  $0.5EI$  and  $0.50EI$  for beams and columns, respectively, are used in this study, [5].

### **5.4 Comparison of responses from the analysis**

In this section, Non-linear Static Pushover analysis has been performed for reinforced building of Linear and L-Shape or Nonlinear with and without shear wall located in Ethiopia first for (Zone 2 and  $PGA=0.05g$ ) plus (Zone 4 and  $PGA=0.1g$ ). In order to investigate the result of code comparison. In additional Response Spectrum analysis also performed. Results of these analyses have been presented and compared to each other. To estimate the seismic demands the contribution of the value of gravitational acceleration of  $PGA= 0.05g$  and  $0.1g$  for both linear and L-shaped or Nonlinear structure with and without shear wall was included in the pushover analysis of the building. The combined values of story shear, floor displacements, pushover curves, performance point and storey drifts were computed include

linear with and without shear wall for both Zone 2 and Zone 4, and also for L-shaped building for both PGA=0.05g and PGA=0.1g. Figure shows the floor displacements, interstorey drift ratio and storey shear demand for both linear and L-shape or Nonlinear B+7 story building with and without shear wall. The values of lateral displacement, inter-storey drifts and shears were computed at the floor levels for the two analysis directions( X and Y) and compared to each other.

#### 5.4.1 Floor Displacements and Inter-Storey Drift Ratios

##### a) Floor Displacements

The floor displacement is given by

$$U_{jxn}(t) = \Gamma_n \phi_{jxn} D_n(t) \qquad U_{jyn}(t) = \Gamma_n \phi_{jyn} D_n(t) \qquad (5.1)$$

##### b) Inter-Storey Drift Ratios

The storey drifts in the x and y directions defined at the CM is given by

$$\Delta_{jxn}(t) = \Gamma_n (\phi_{jxn} - \phi_{j-1,xn}) D_n(t) \qquad \Delta_{jyn}(t) = \Gamma_n (\phi_{jyn} - \phi_{j-1,yn}) D_n(t) \qquad (5.2)$$

The inter-storey drift ratio (%) is given by

$$\Delta_{jxn}(t)/h_j \times 100 \qquad \Delta_{jyn}(t)/h_j \times 100 \qquad (5.3)$$

Where the subscripts x and y show the responses are in X and Y direction respectively. On the structural level, the inter-storey drift ratio is one of the simplest and most essential damage indicators.

Table 5.11 Inter Storey drift limits for different performance level[12]

Inter-storey Displacement Limits	Performance Level		
	Immediate Occupancy	Life Safety	Collapse Prevention
$(\Delta_i / h_i) \times 100$	1.0	2.0	3.0

### 5.4.2 Requirement for serviceability limit state

According to EBCS-8 [12] section 2.4.3.2. For building having non-structural elements of brittle materials attached to the structure, the requirements for serviceability limit states is considered satisfied if the inter-storey drifts(%) are limited to :

$$d_r/h \times 100 \leq 1.0 \quad (5.4)$$

Where:  $d_r$  = design inter-storey drift, evaluation difference of the average lateral displacements at the top and bottom of the storey under consideration, and

$h$  = Storey height

Therefore the maximum allowable design inter-storey drift ratio(%) for both regular 8 storey building and b+7 storey irregular building is given as:

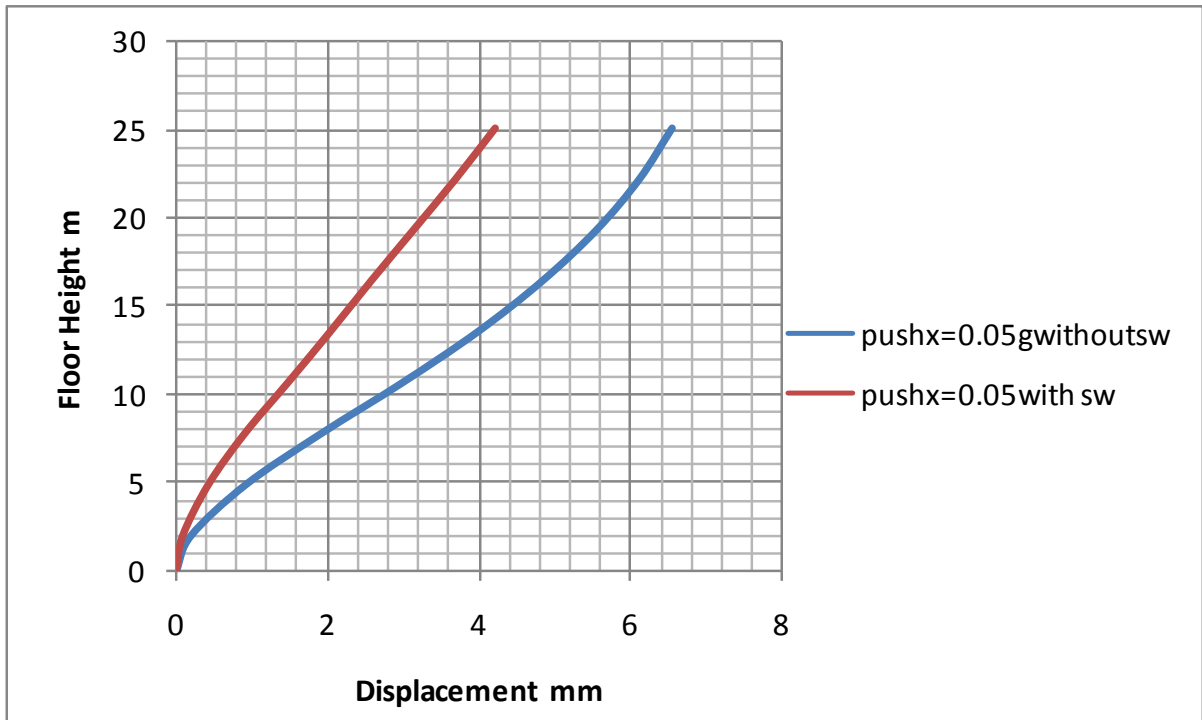
$$d \leq 1.0 \text{ for serviceability limit state}$$

### 5.4.3 Comparison of floor displacements

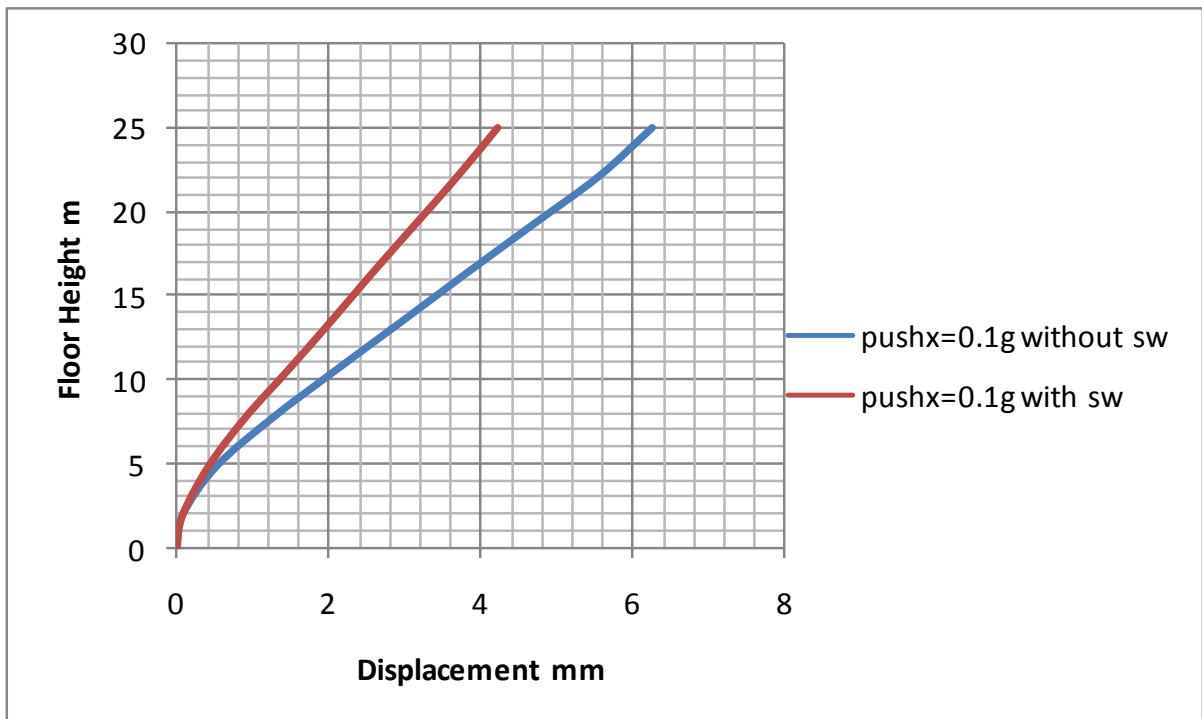
In order to assess various analysis methods, the variations of displacements along the height of the building using the specified methods of analysis are obtained and compared with out another. Floor displacement for G+7 linear and L-Shaped with and without shear wall have been shown and also the two gravitational acceleration coefficient that is  $PGA = 0.05g$  and  $0.1g$  included in the comparison. The result of floor displacement are shown in figure below 5.1,5.2,5.3 and 5.4 respectively. The displacement decreases when shear wall exists in the building, the displacement verses floor height graph displays the increases of displacement as the floor height increases. And the displacement is comparatively high on the Y-direction relative to the X-direction, this all is similar for the old and new Ethiopian building code, only with the difference of some values with respect to gravitational acceleration values.

The liner building verses the non-linear building comparatively displays the displacement values is higher on non linear building both in X and Y-direction.

The presence of shear wall in non linear building does not shows a comparative visible difference on the displacement as per the storey height over the Y-direction.

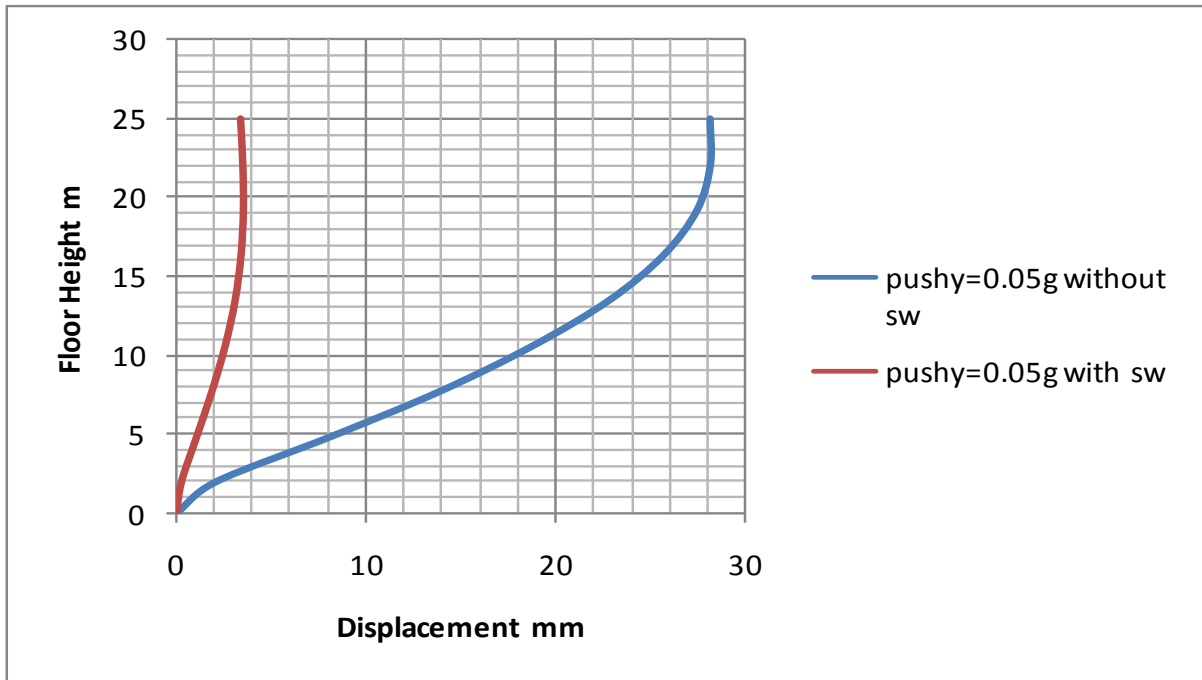


a) In X-direction (0.05g)

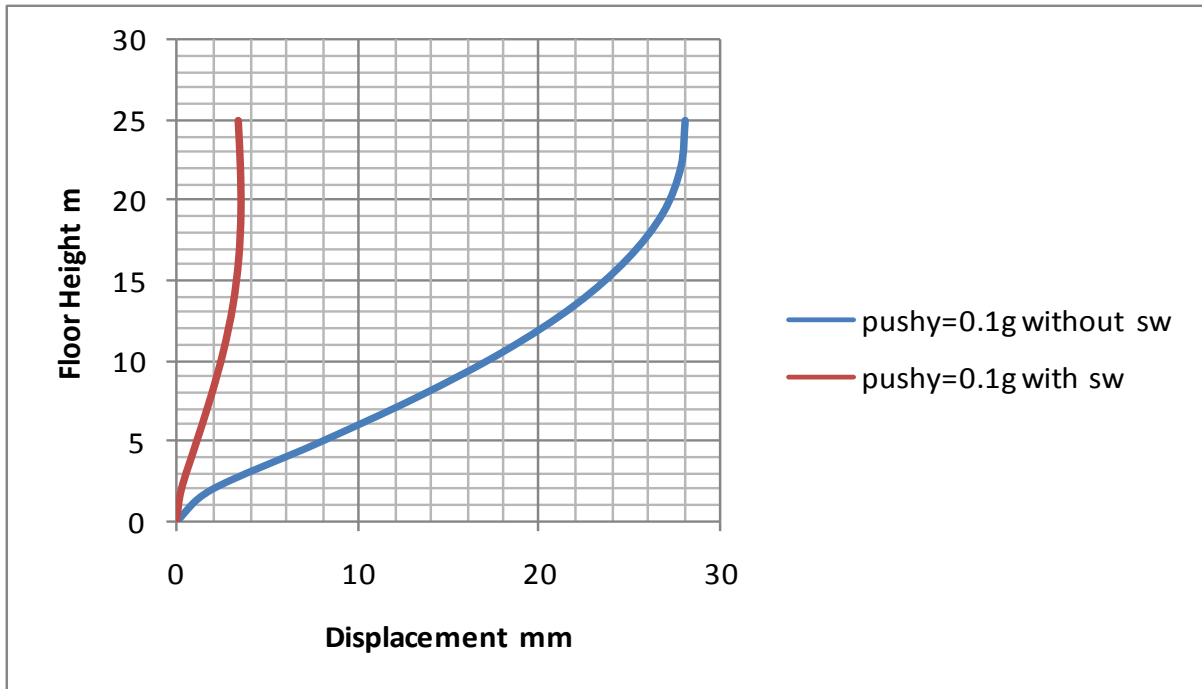


b) In X-direction(0.1g)

Figure 5.0. Storey displacement profile for eight storey RC linear building with and without shear wall (According to Old Ethiopian building code)

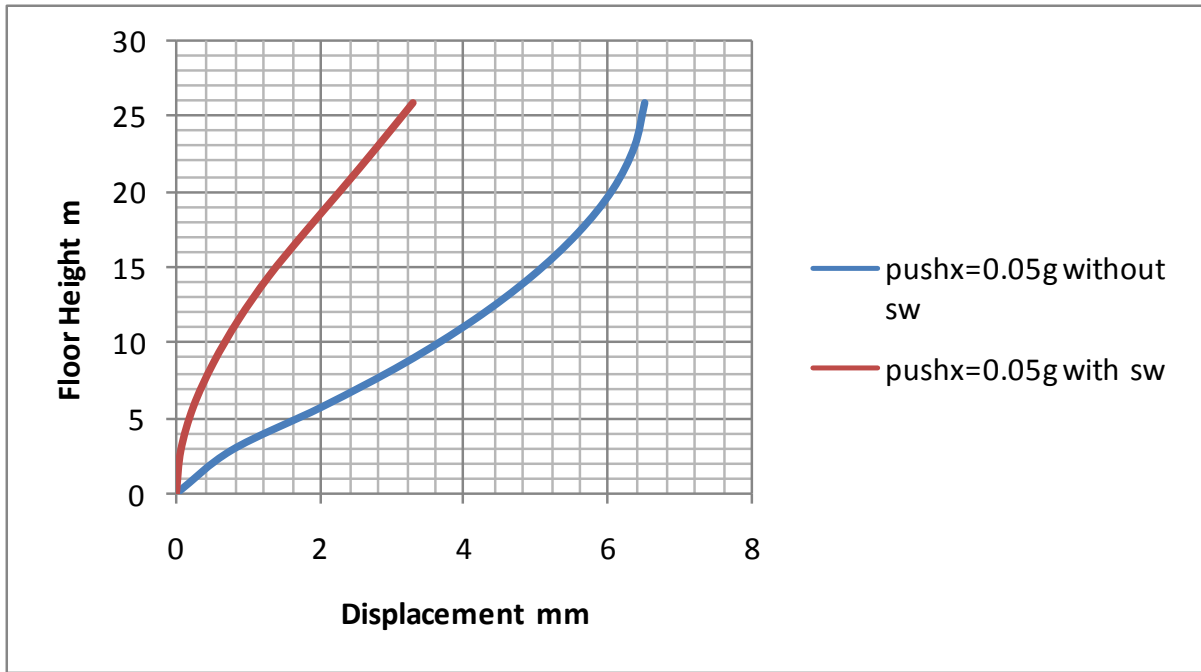


a) In Y-direction (0.05g)

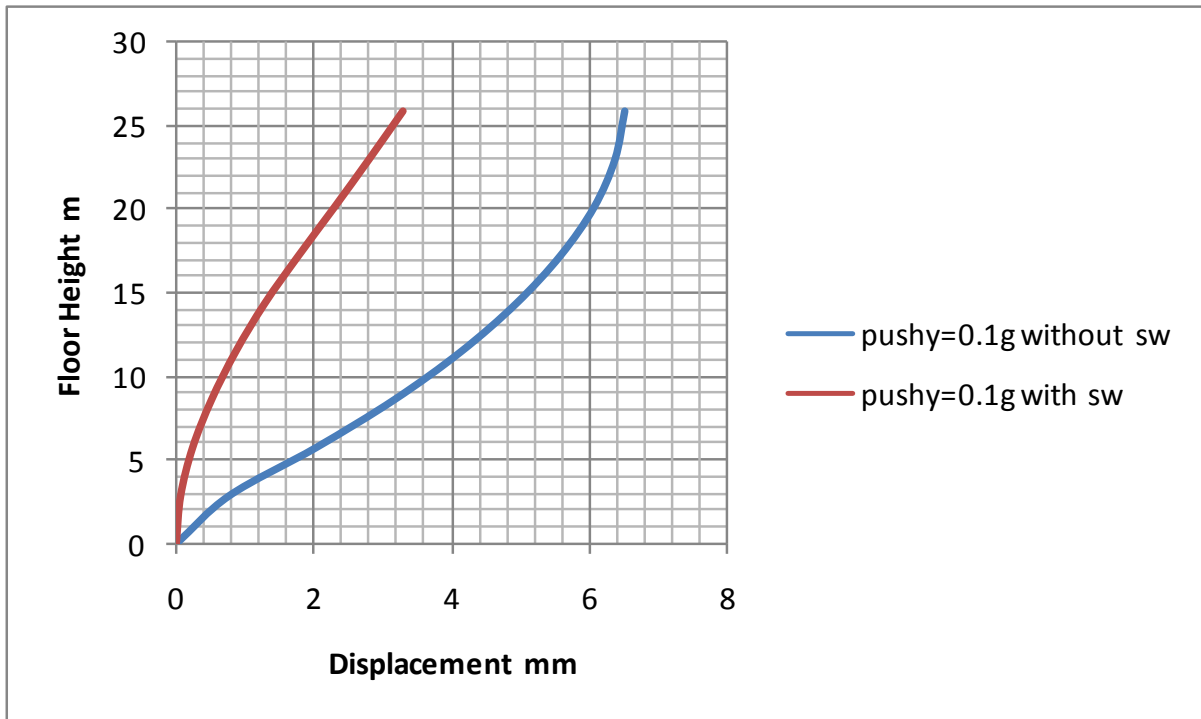


b) In Y-direction (0.1g)

Figure 5.1. Storey displacement profile for eight storey RC linear building with and without shear wall (According to New Ethiopian building code)

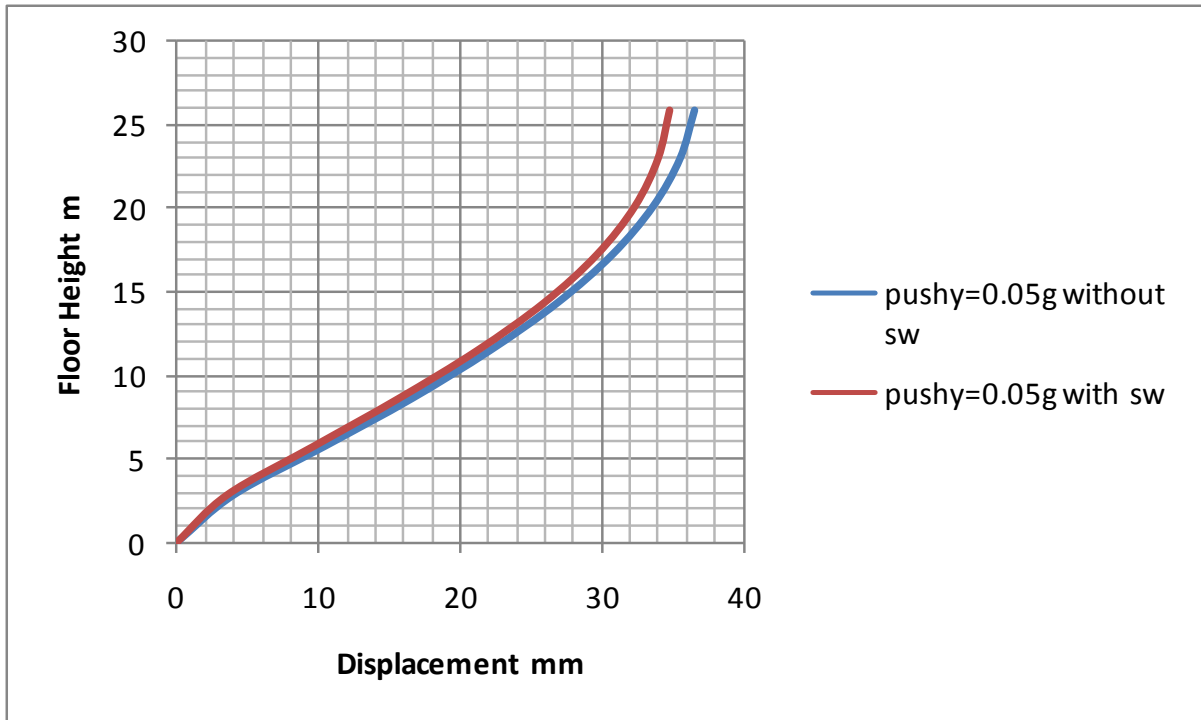


a) In X-direction (0.05g)

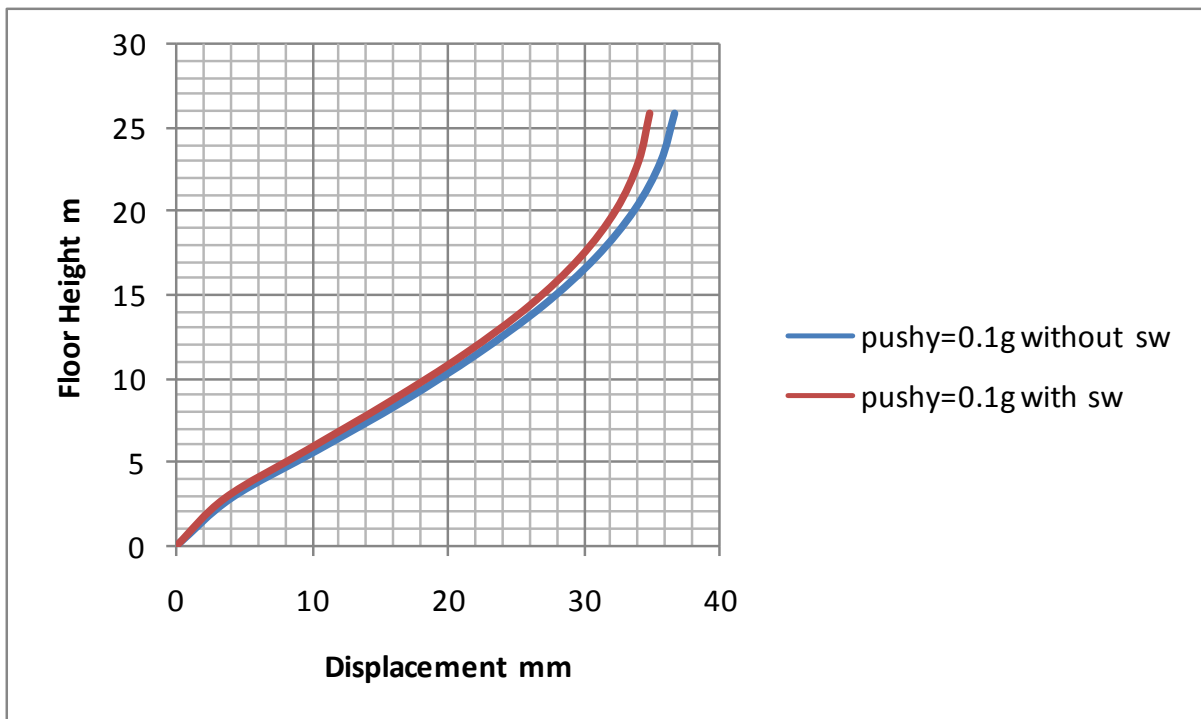


b) In X-direction (0.1g)

Figure 5.2. Storey displacement profile for eight storey RC L-shaped or nonlinear building with and without shear wall (According to Old Ethiopian building code)



a) In Y- direction (0.05g)



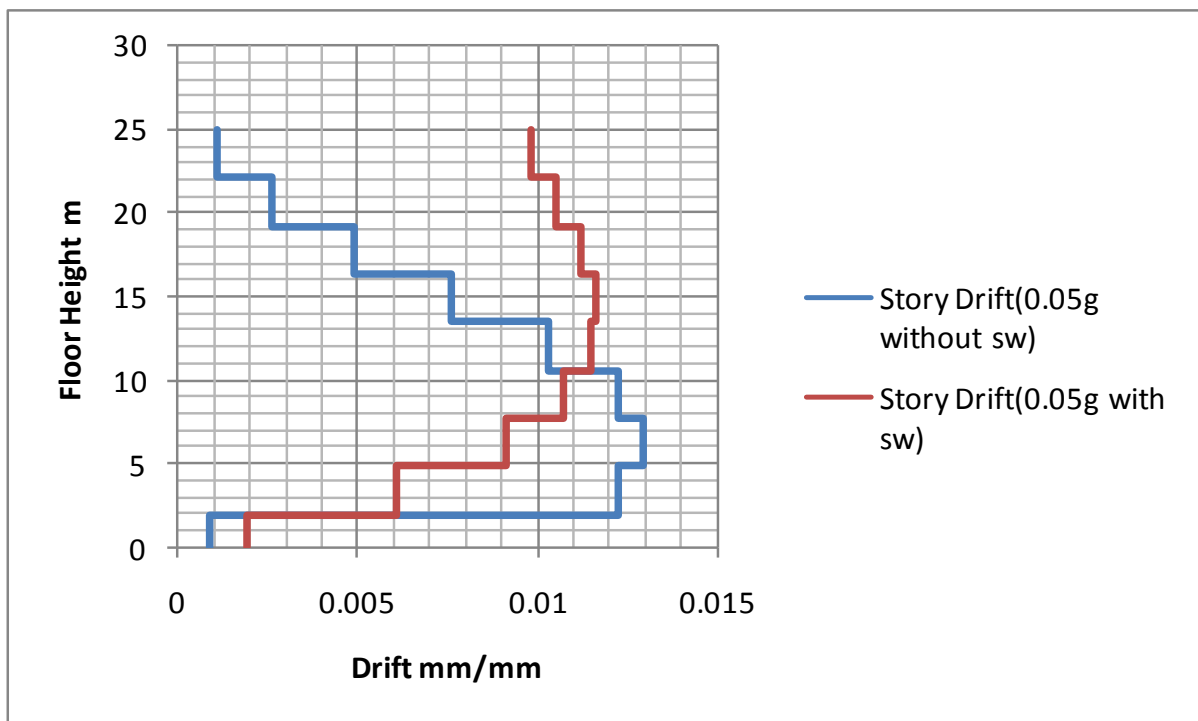
b) In Y- direction (0.1g)

Figure 5.3. Storey displacement profile for eight storey RC L-shaped or nonlinear building with and without shear wall (According to New Ethiopian building code)

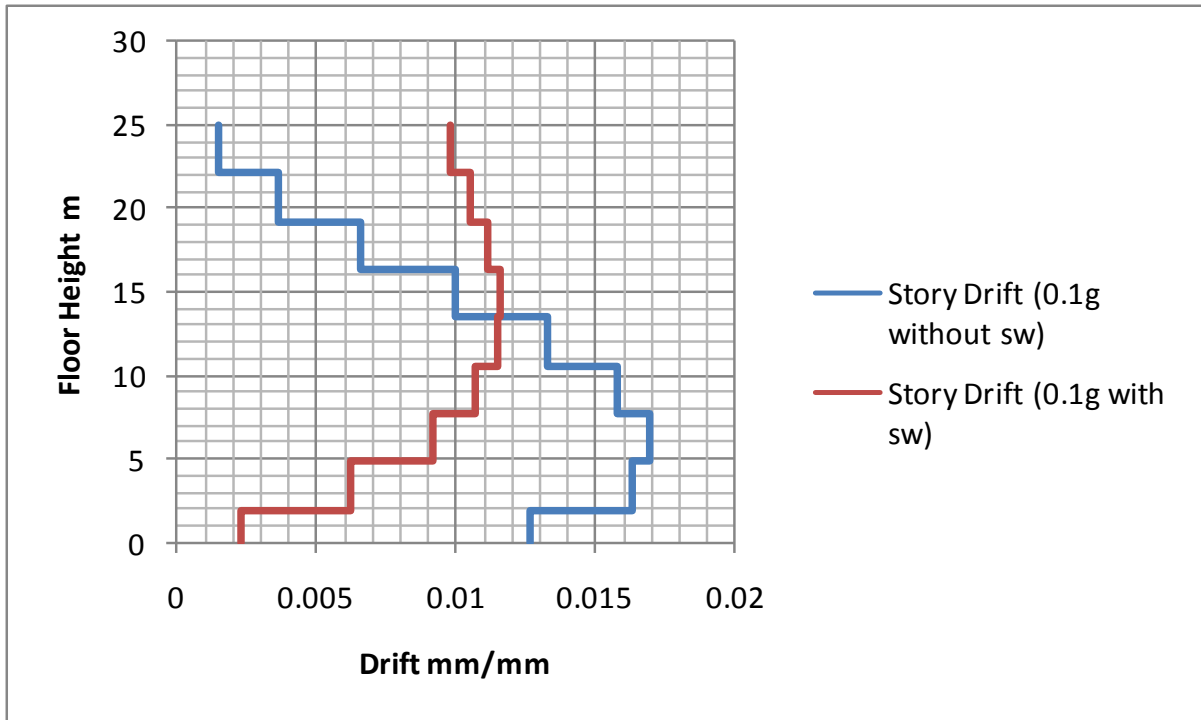
#### 5.4.4 Comparison of Inter-Storey drifts Ratios

The inter-storey drift ratio is one of the simplest and most essential damage indicators of the structure. The Inter-Storey drift limit for different performance level is shown in table 5.1.3. Inter-Storey ratio for Linear, L-Shaped 7 storey building with and without shear wall and also for gravitational acceleration coefficient for both old and new Ethiopian building code( i.e PGA= 0.05g and 0.1g ) are shown in the figure for the specified methods of analysis under consideration. The result of inter-storey drift shown in the graph shown below 5.5,5.6,5.7 and 5.8 respectively, the inter-storey drift value verses the floor height displays the non linear relationship of floor height and inter-storey drift value.

The inter-storey drift value is higher on the Y-direction both in linear and non-linear buildings, this all is similar for both the old and new Ethiopian building code, only with the difference of some numerical values with respect to the peak ground gravitational acceleration values.

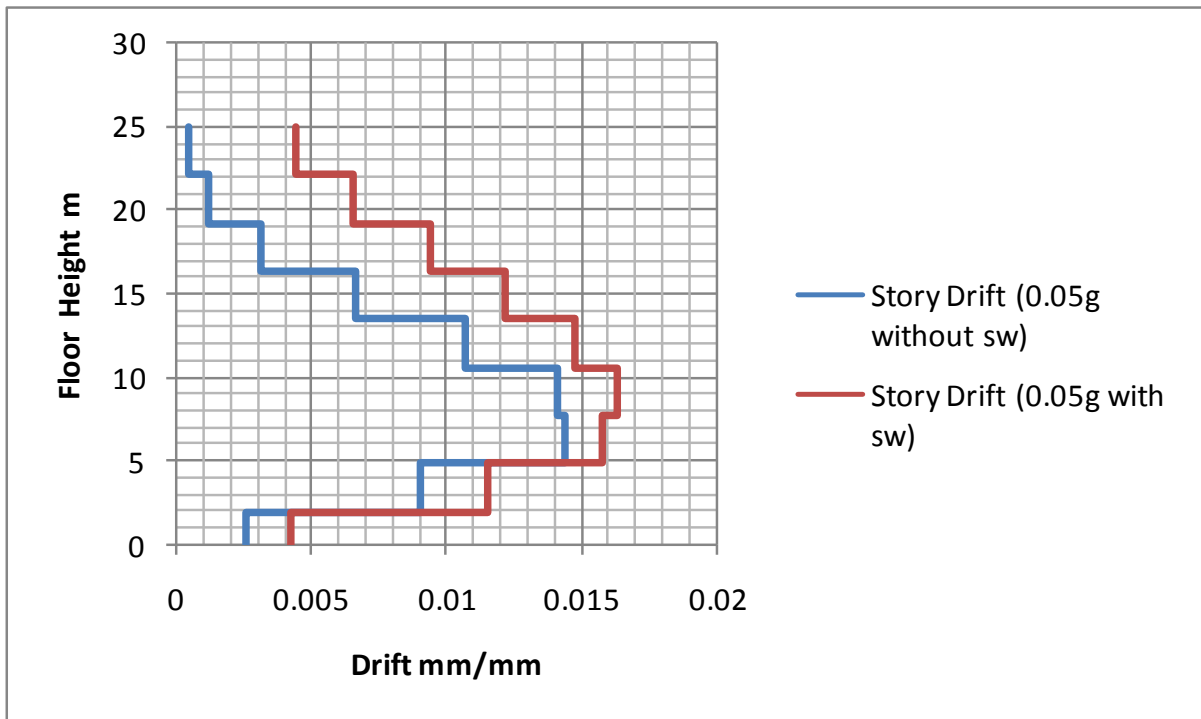


a) In X-direction (0.05g)

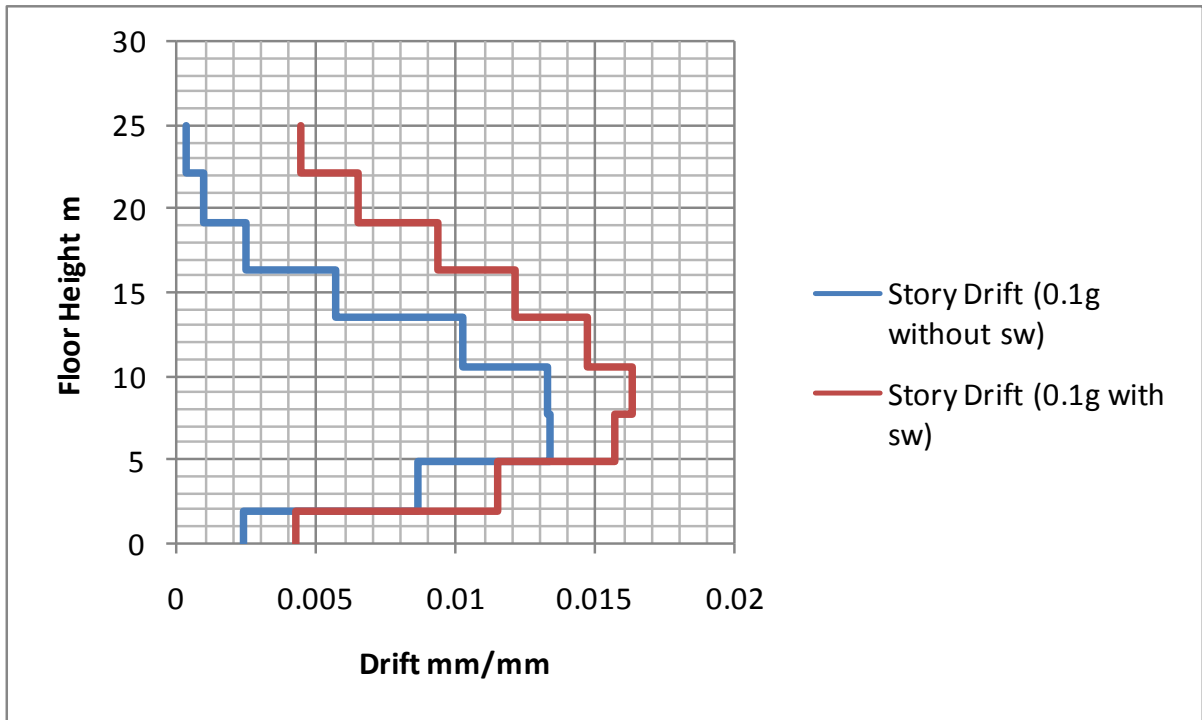


b) In X-direction(0.1g)

Figure 5.4. Storey drifts profile for eight storey RC linear building with and without shear wall (According to Old Ethiopian building code)

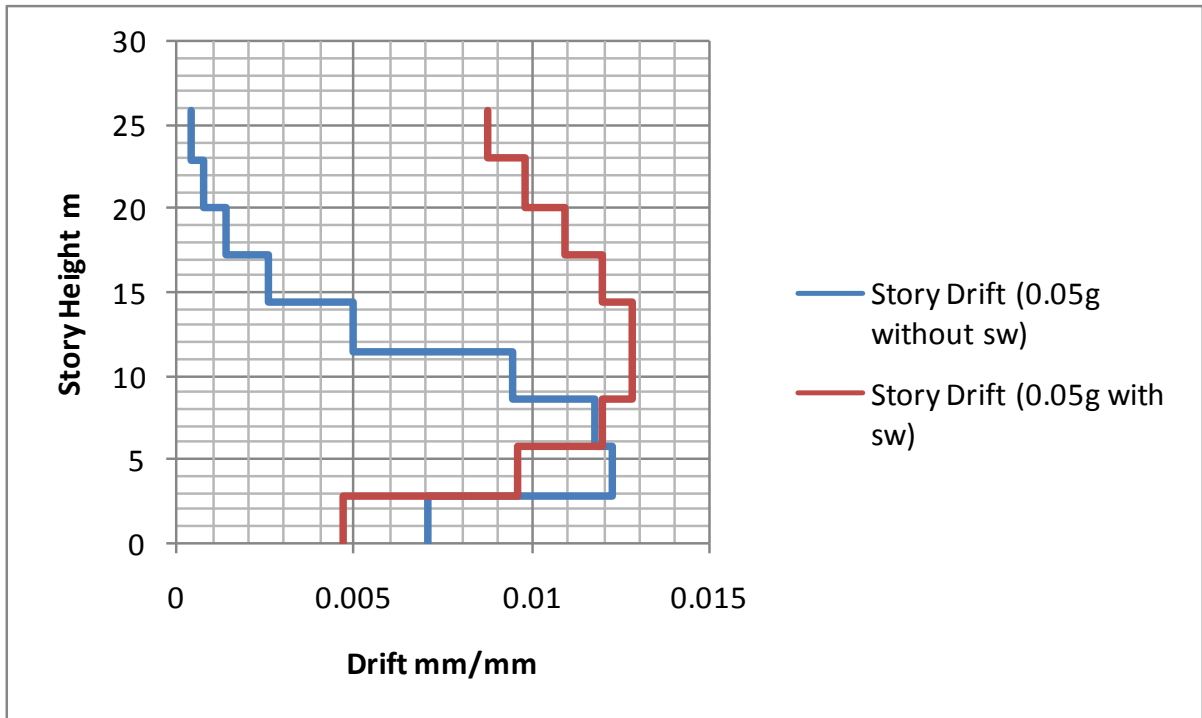


a) In Y-direction(0.05g)

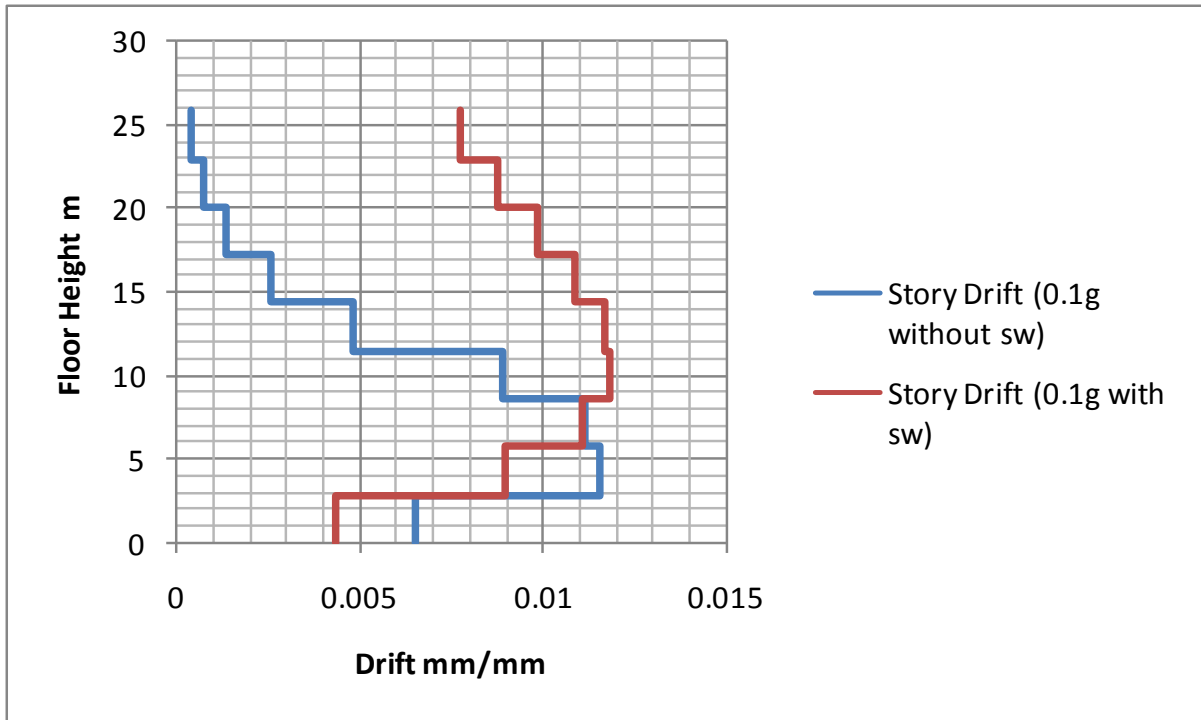


b) In Y-direction(0.1g)

Figure 5.5. Storey drifts profile for eight storey RC linear building with and without shear wall (According to New Ethiopian building code)

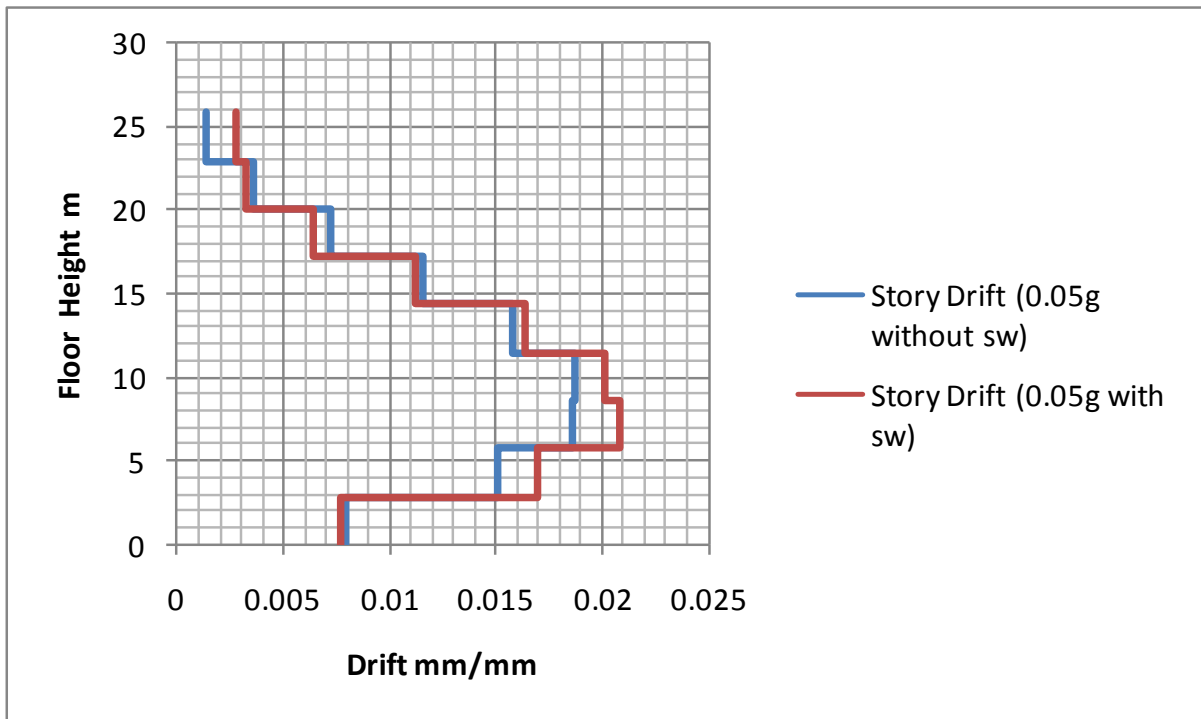


a) In X-direction(0.05g)

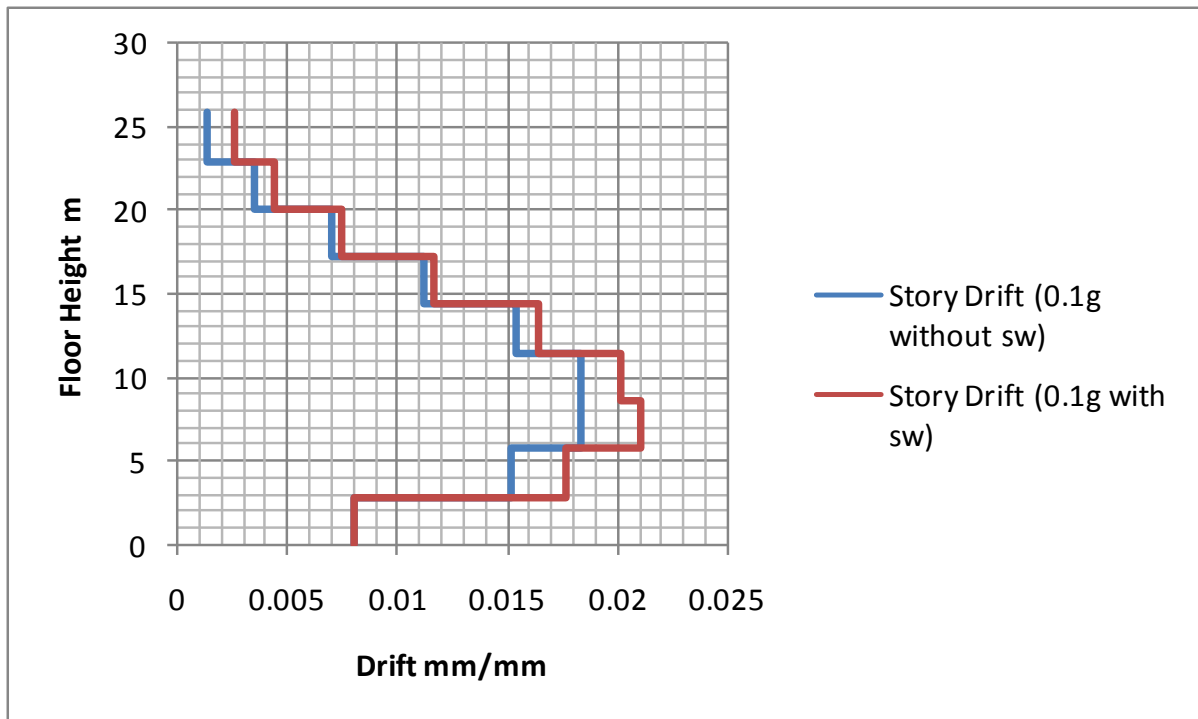


b) In Y-direction(0.1g)

Figure 5.6. Storey drifts profile for eight storey RC L-shaped or nonlinear building with and without shear wall (According to Old Ethiopian building code)



a) In Y-direction (0.05g)



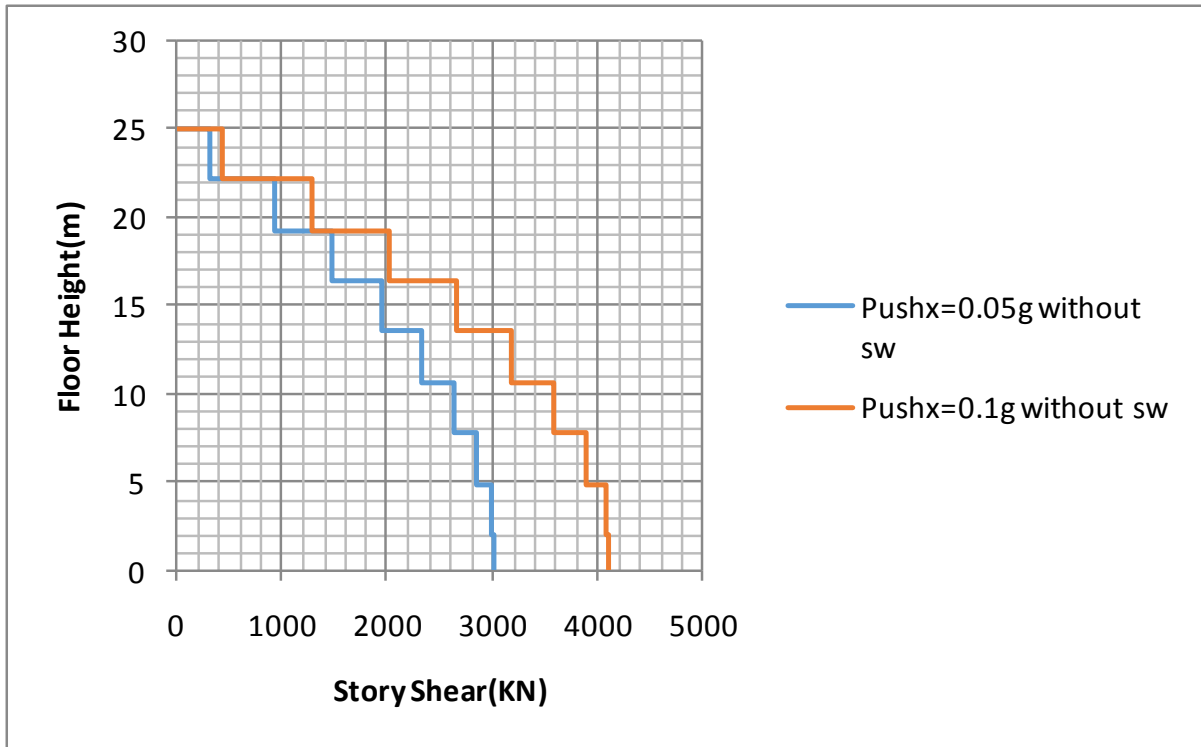
b) In Y-direction (0.1g)

Figure 5.7. Storey drifts profile for eight storey RC L-shaped or nonlinear building with and without shear wall (According to Old Ethiopian building code)

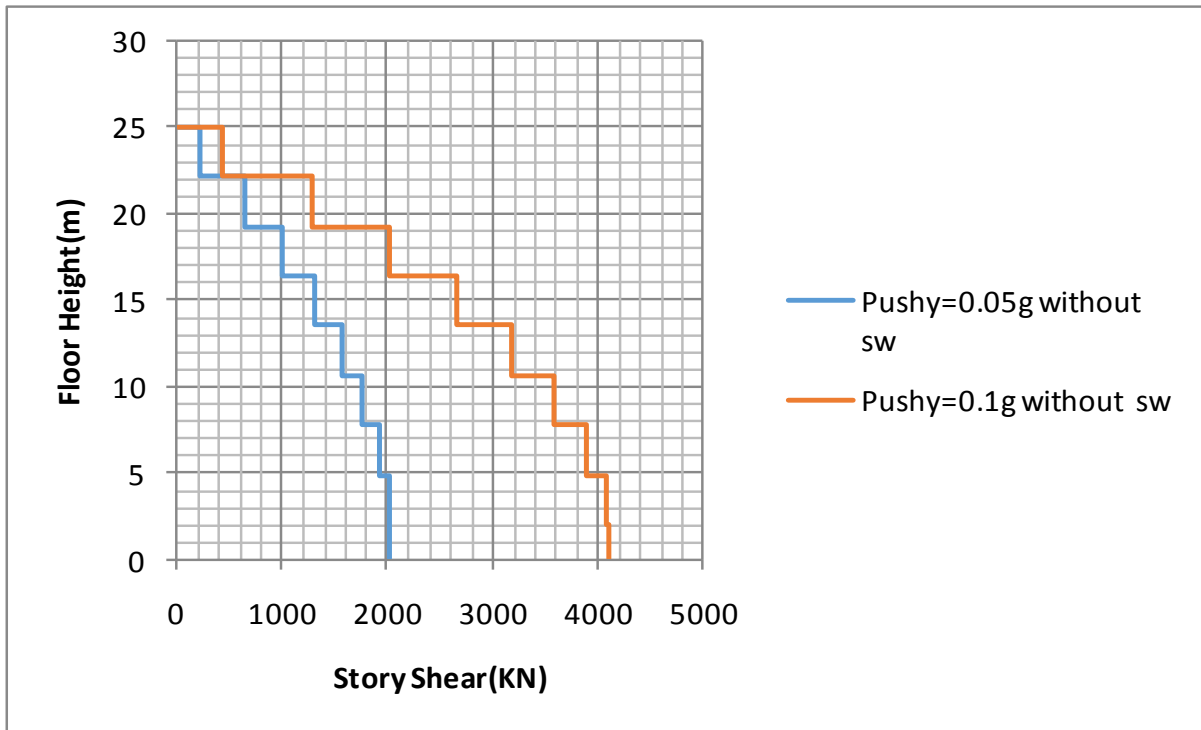
### 5.5 Storey Shear Comparison

In this section, the storey shear values obtained for the pushover analysis of Linear, L-Shaped seven storey building with and without shear wall and also including the two value of gravitational acceleration coefficient for both old and new Ethiopian building code were compared. The result of shear forces in the graph below 5.9.5.10, 5.11 and 5.12 respectively shows the shear force decreases with the presence of shear wall in the building. The shear forces value is almost doubled as the value of peak ground acceleration is doubled ( peak ground acceleration of new Ethiopian building code is twice of that of the old code).

The storey shear is lower on non-linear building as compared to the linear building both in X and Y-direction of the old and new Ethiopian building code values of the peak ground acceleration.

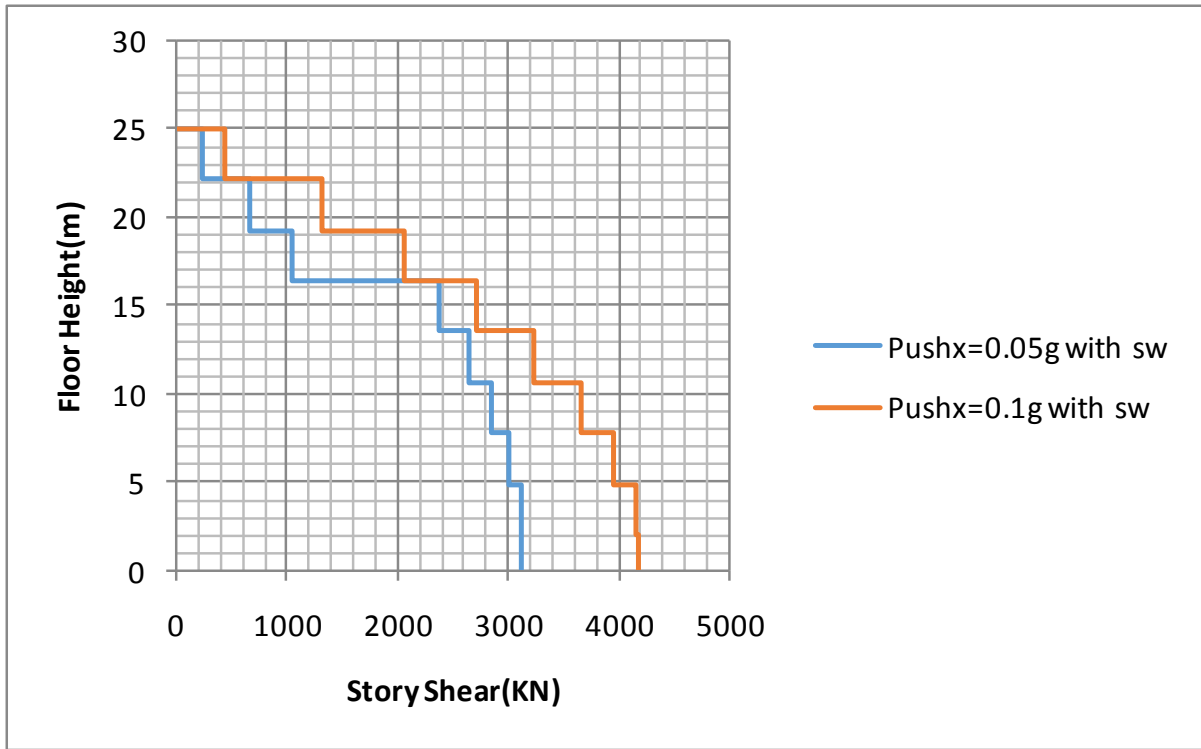


a) In X-direction (0.05g and 0.1g)

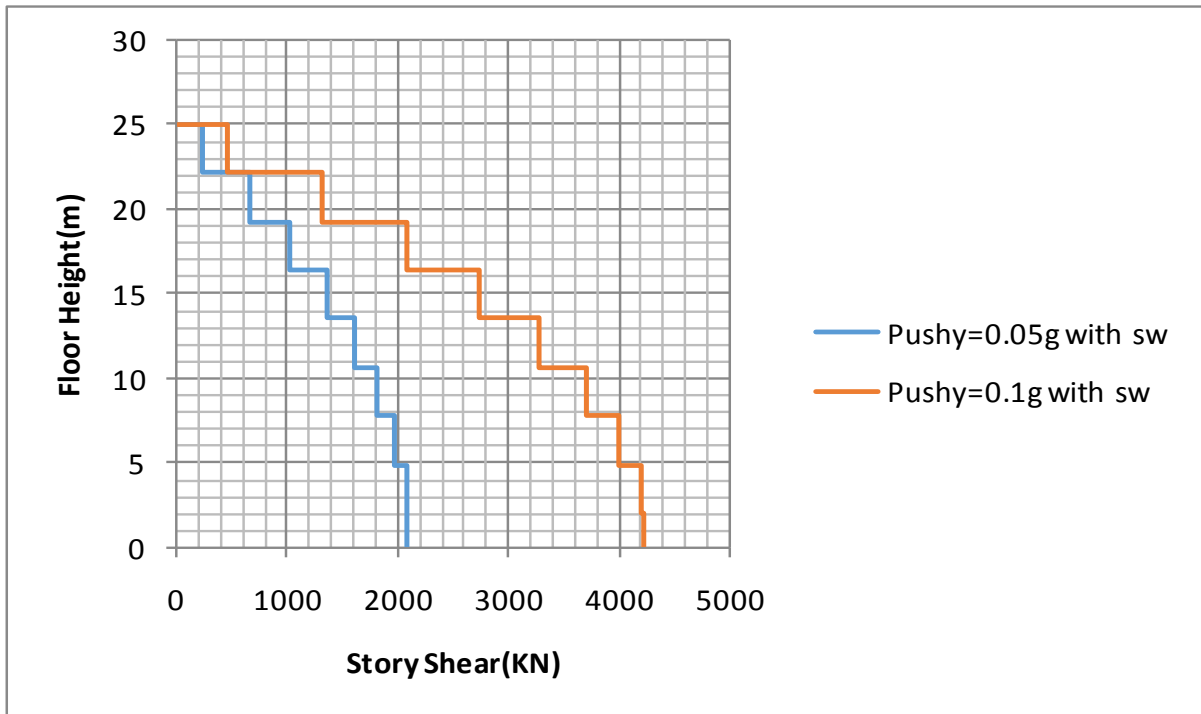


b) In Y-direction (0.05g and 0.1g)

Figure 5.8. Storey shear force profile for eight storey RC linear with shear wall (According to New and Old Ethiopian building code)

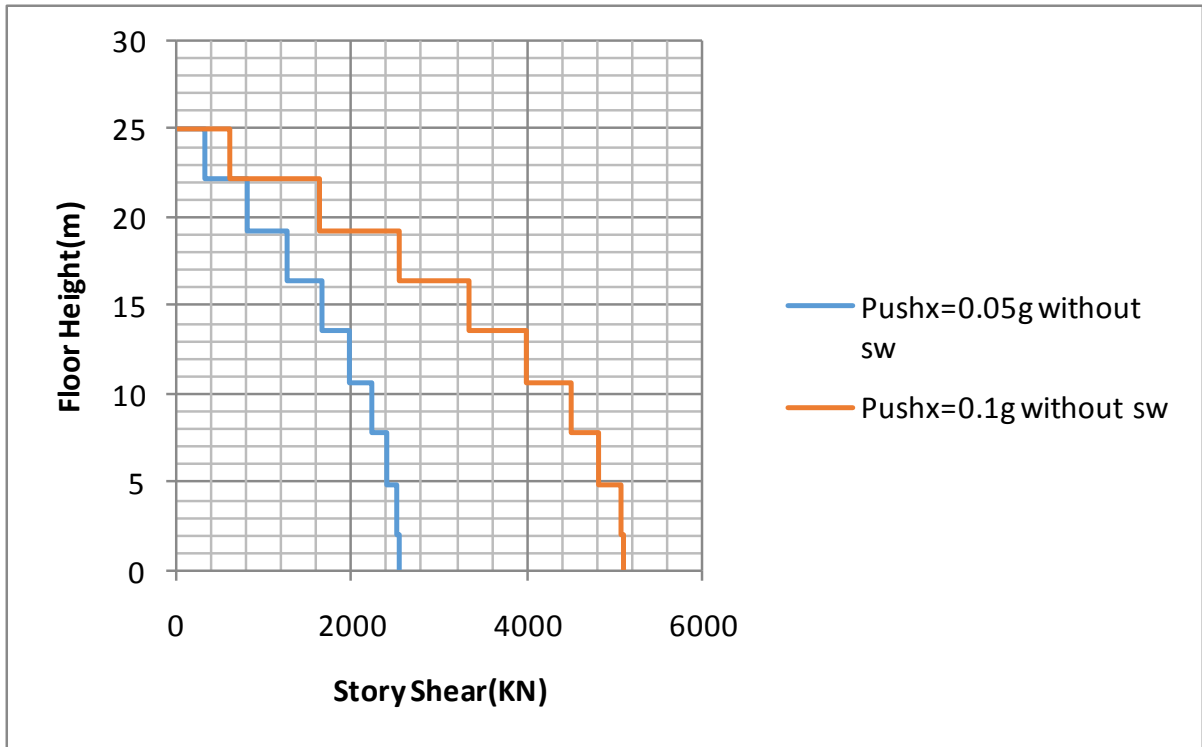


a) In X-direction( 0.05g and 0.1g)

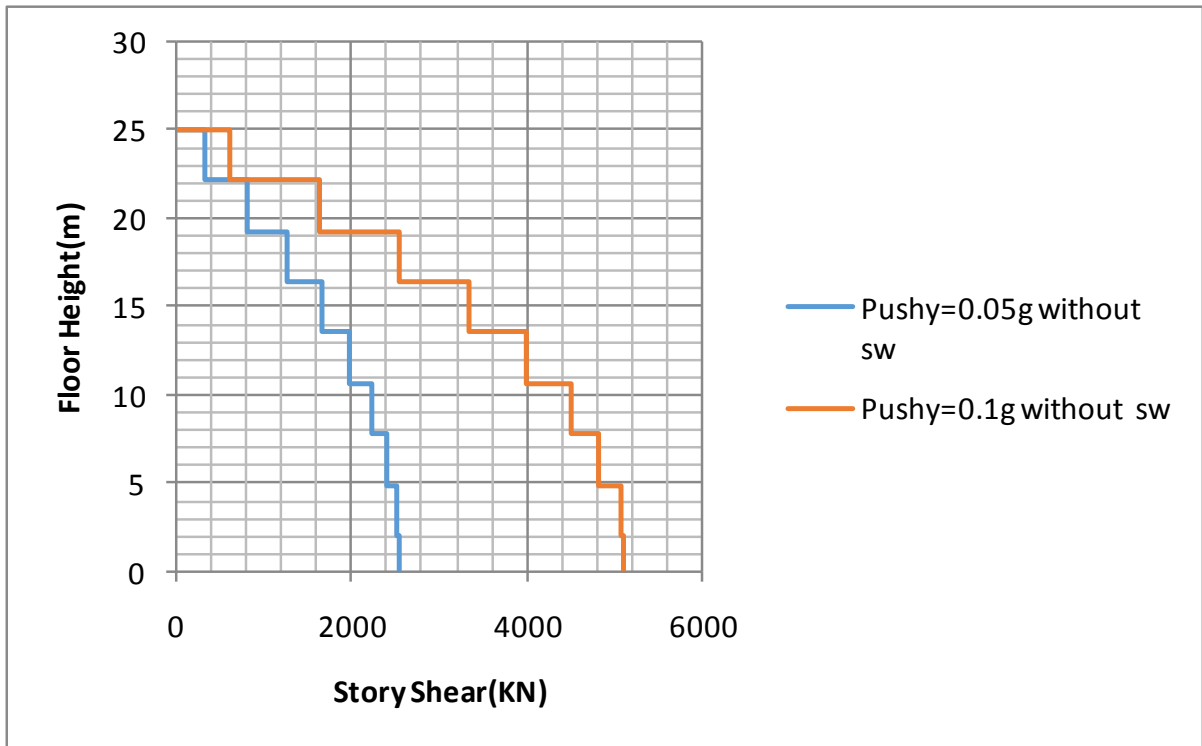


b) In X-direction (0.05g and 0.1g)

Figure 5.9. Storey shear force profile for eight storey RC linear with shear wall (According to New and Old Ethiopian building code)

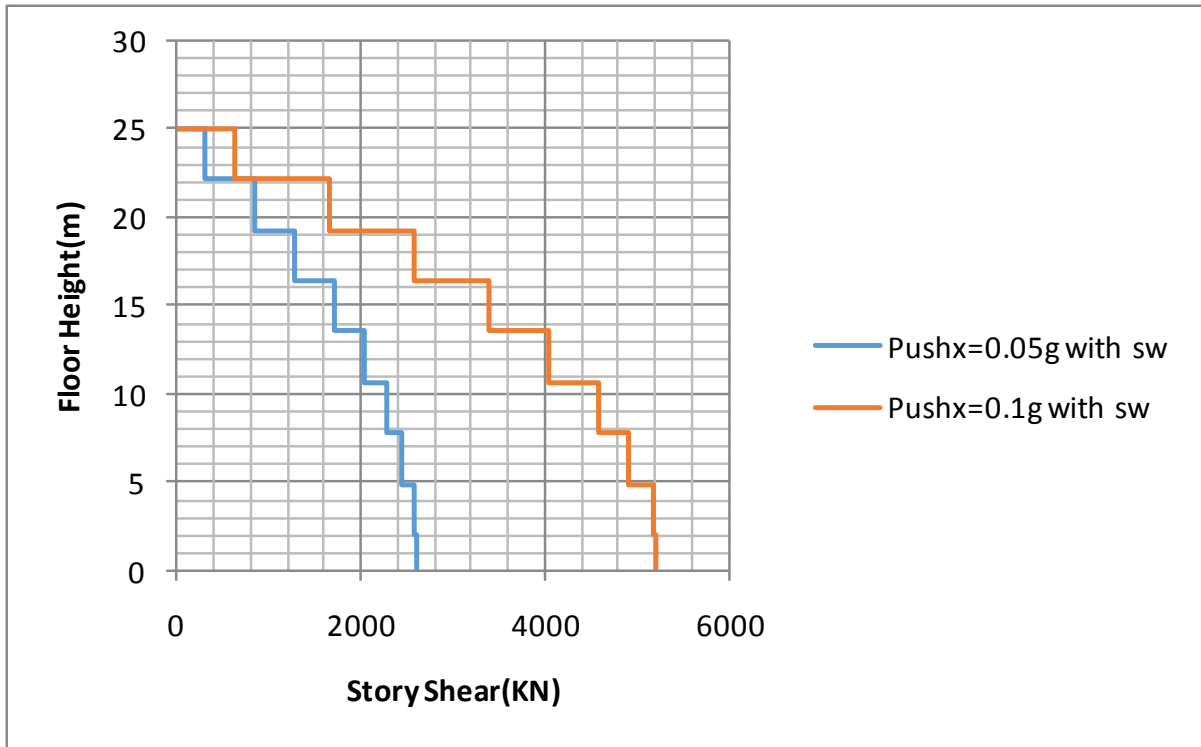


a) In X-direction (0.05g and 0.1g)

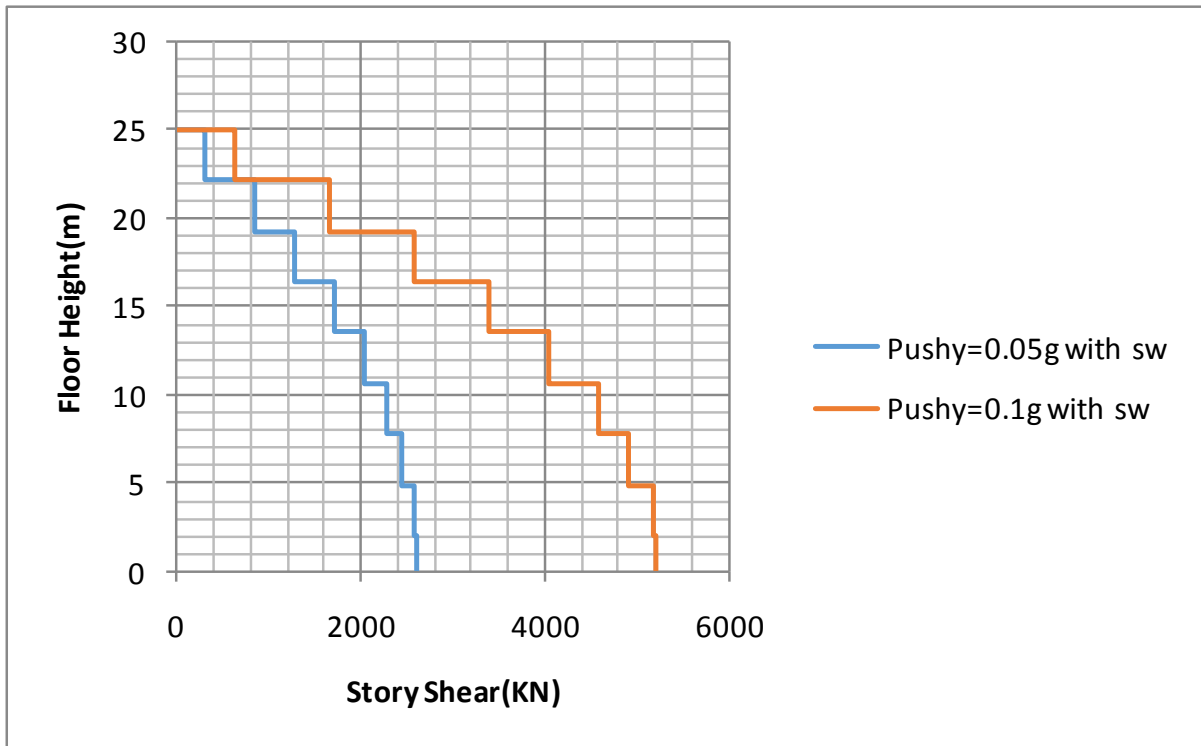


b) In X-direction ( 0.1g)

Figure 5.10. Storey shear force profile for eight storey RC L-shaped or nonlinear without shear wall (According to New and Old Ethiopian building code)



a) In X-direction (0.05g and 0.1g)



b) In Y-direction ( 0.1g)

Figure 5.11. Storey shear force profile for eight storey RC L-shaped or nonlinear with shear wall (According to New and Old Ethiopian building code)

## **5.6 Plastic Hinge Formation**

Even though, asymmetrical structures subjected to pair of actions at a time when subjected to horizontal earthquake forces, pushover analysis is carried out only in the direction of larger displacement.

## **5.7 Frame Level Verification**

### **5.7.1 Verification of SAP2000**

The following frame is a two bay, three story frame taken from Ahmet Yakut (2001) from his numerical study. It is going to be analyzed using hand calculation of nonlinear static pushover analysis in which an incremental load is applied to the structure at each story and the corresponding internal moments are recorded and the load is applied until hinge is formed and final a progressive failure is achieved.

#### **5.7.1.1 Basic Assumptions**

- Constant Axial Load on Columns for Analysis Steps
- Rigid-plastic with no hardening or softening moment-rotation behaviour for columns and beams.
- Plastic hinging occurs when moment capacity is within 5% tolerance.

#### **Analysis Procedure**

- Create the model using SAP2000 software like any other modelling.
- Define member properties for both beams and columns and determine the section capacity and moment curvature relation.
- Apply gravity load on the frame elements, no lateral load is applied her.
- Define the load to be applied, it can be a rectangular , inverted triangle or the first mode shape can be used. Inverted triangle load distribution method is used.
- Apply the lateral load incrementally. Find the lateral load that will induce the first hinge point and once the first hinge is formed elastic analysis is no applicable; record the load effects due to this lateral load at each element.
- Apply an actual hinge at the location of first hinge point by assigning a frame release option then remove the vertical load and apply an incremental lateral load on the frame so that the new load effect on each element and the load from the previous

action will result in a new hinge at a new location. Continue this procedure until collapse or mechanism.

- The base shear is the sum of incremental load at each step. Plot the base shear verses the top displacement this will results in the capacity curve.

### 5.7.1.2 Frame Verification Data

Figure 5.12 shows the element allocation with its joint assignment of the frame elements.

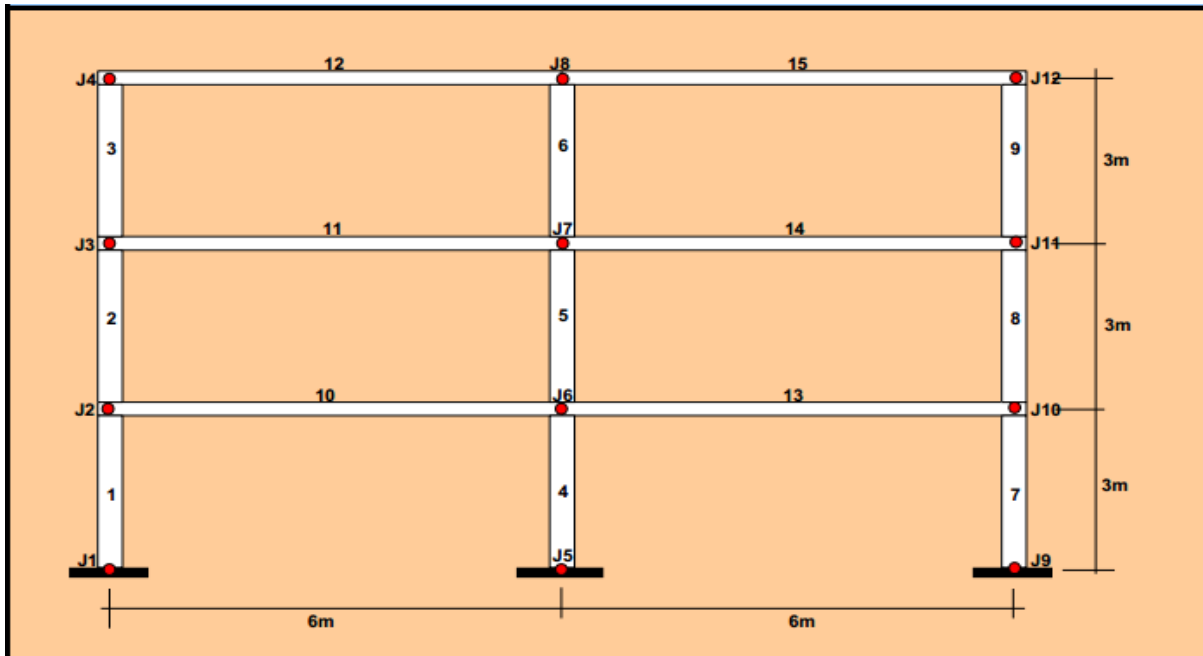


Figure 5.12 Frame element and joint allocation

### 5.7.1.3 Verification frame Loading and Modelling

#### 5.7.1.3.1 Loading on SAP2000

The frame is loaded with gravity load and lateral load. A super imposed dead load of magnitude 10KN/m at third story and 15KN/m at first and second story and service live load or 2KN/m on every story. The lateral load is not known at first so it is increased at every step until mechanism is formed. For the pushover analysis the following loading is used  $DL+0.3LL+EQ$  and it is named “PUSH”.

#### Material Property

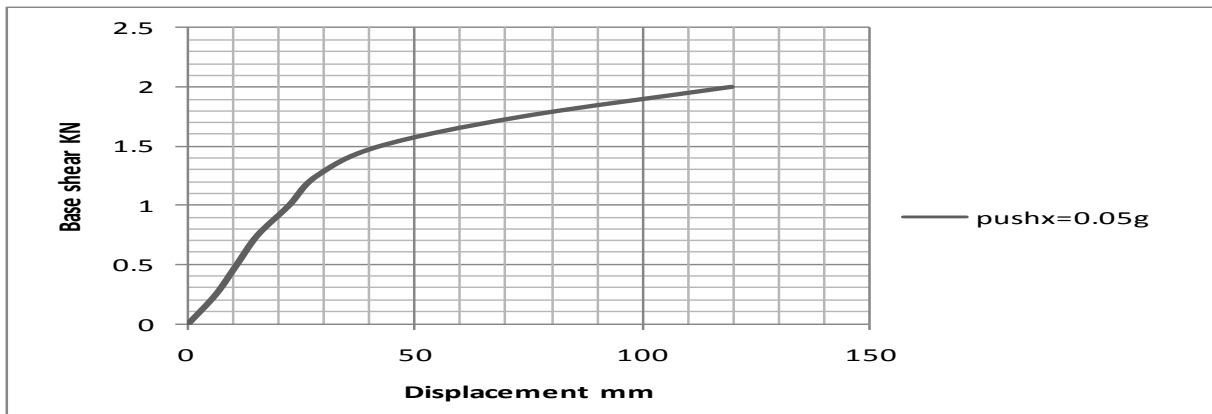
- ❖ C-55 concrete and S-570 steel is used and concrete cover of 50mm is used.
- ❖ Beam(25cmx50cm) ,6Φ10 reinforcement and
- ❖ Column(60cmx60cm),10Φ10 reinforcement

### 5.8 Pushover Curves for Selected RC Buildings

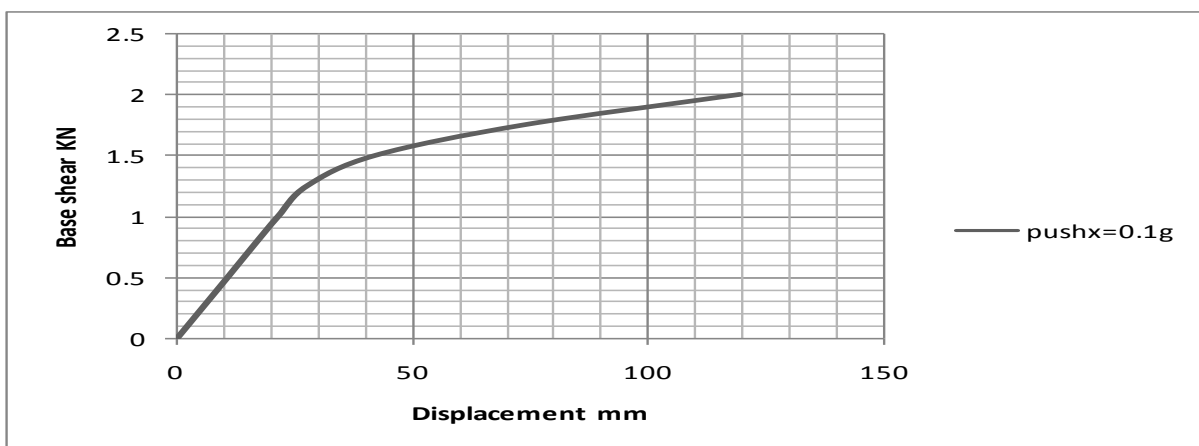
The result of pushover curve shown in the graph below 5.14, 5.15,5.16,5.17,5.18,5.19,5.20 and 5.21 respectively, shows the displacement verses base shear, where the displacement value for the old Ethiopian building code is doubled in the new Ethiopian building code due to the increase of peak ground acceleration values.

The graph pattern of non-linear building shows the curves is closer to linear straight line both in the X and Y-direction, but for linear building the curve is far from straight line.

The presence of shear wall with in the building significantly reduced both the displacement and base shear both in X and Y direction of the linear and non-linear building.

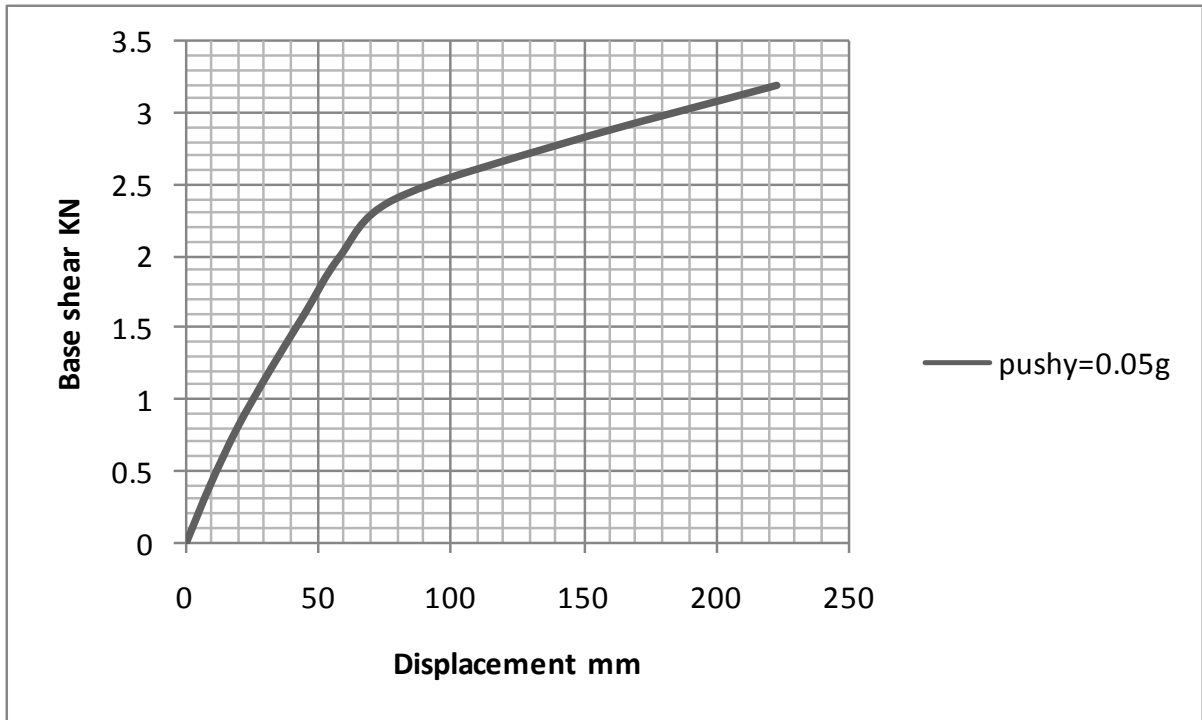


a)

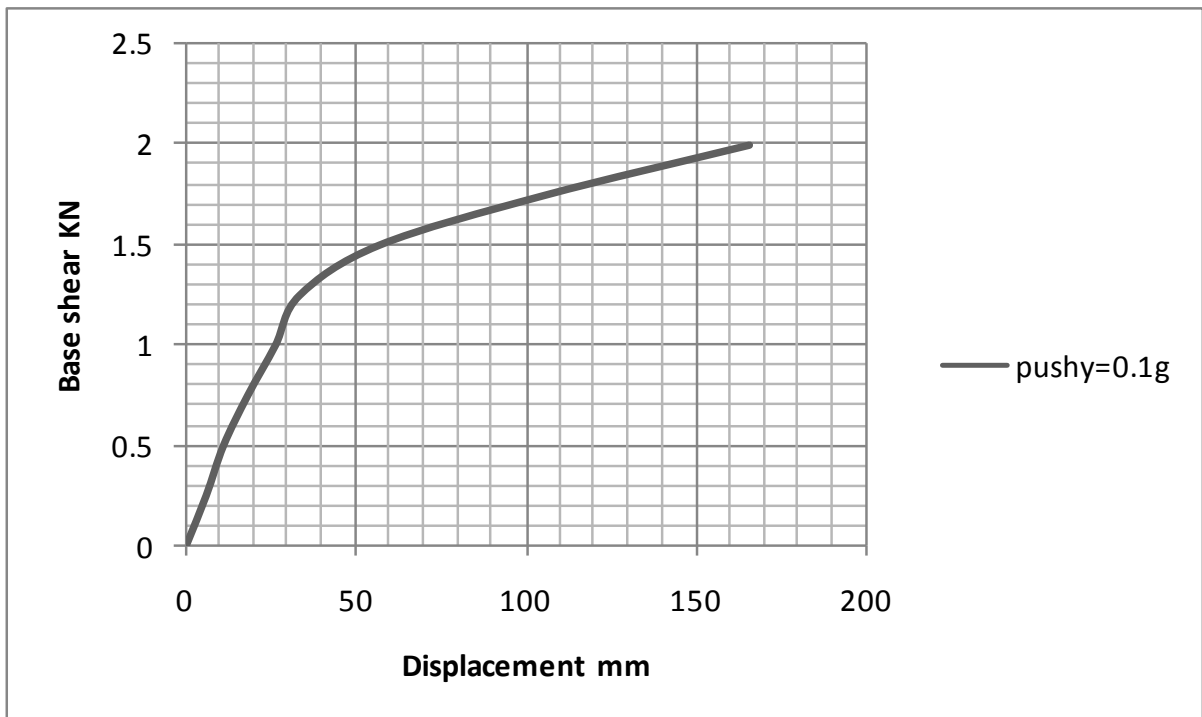


b)

Figure 5.13. Pushover curves of eight storey RC linear building without shear wall a) In X-direction (0.05g) b) In X-direction (0.1g)



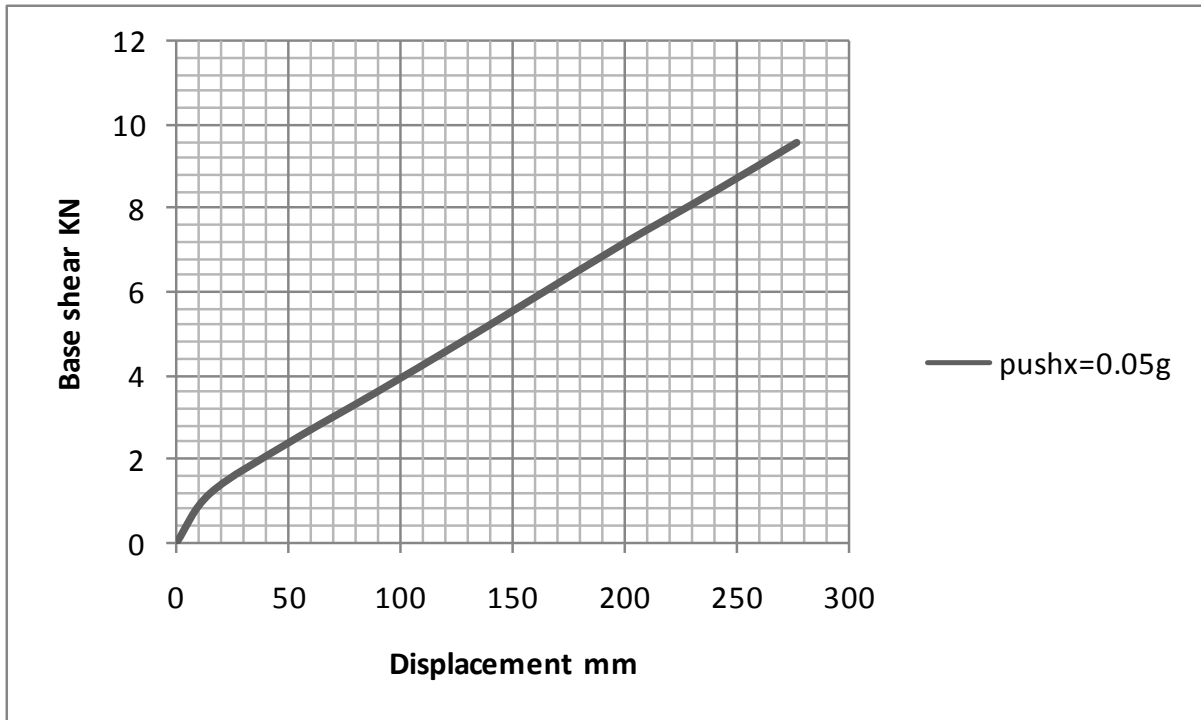
a)



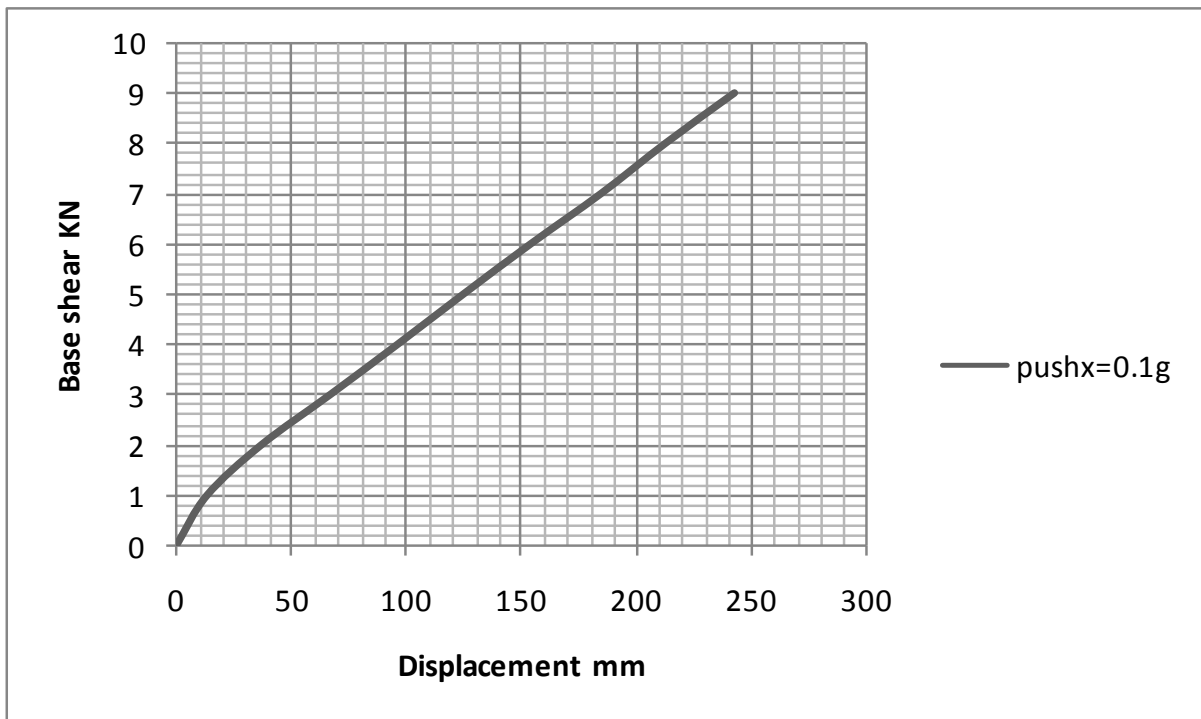
b)

Figure 5.14. Pushover curves of eight storey RC linear building without shear wall

a) In Y-direction (0.05g) b) In Y-direction (0.1g)

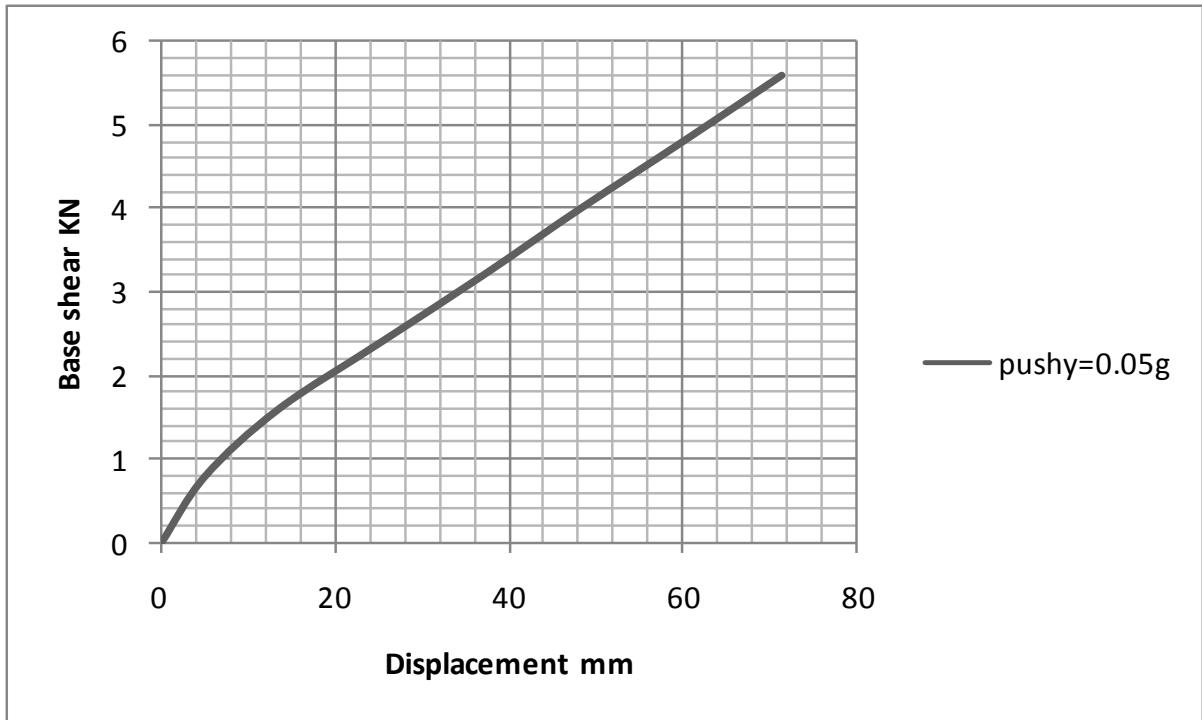


a)

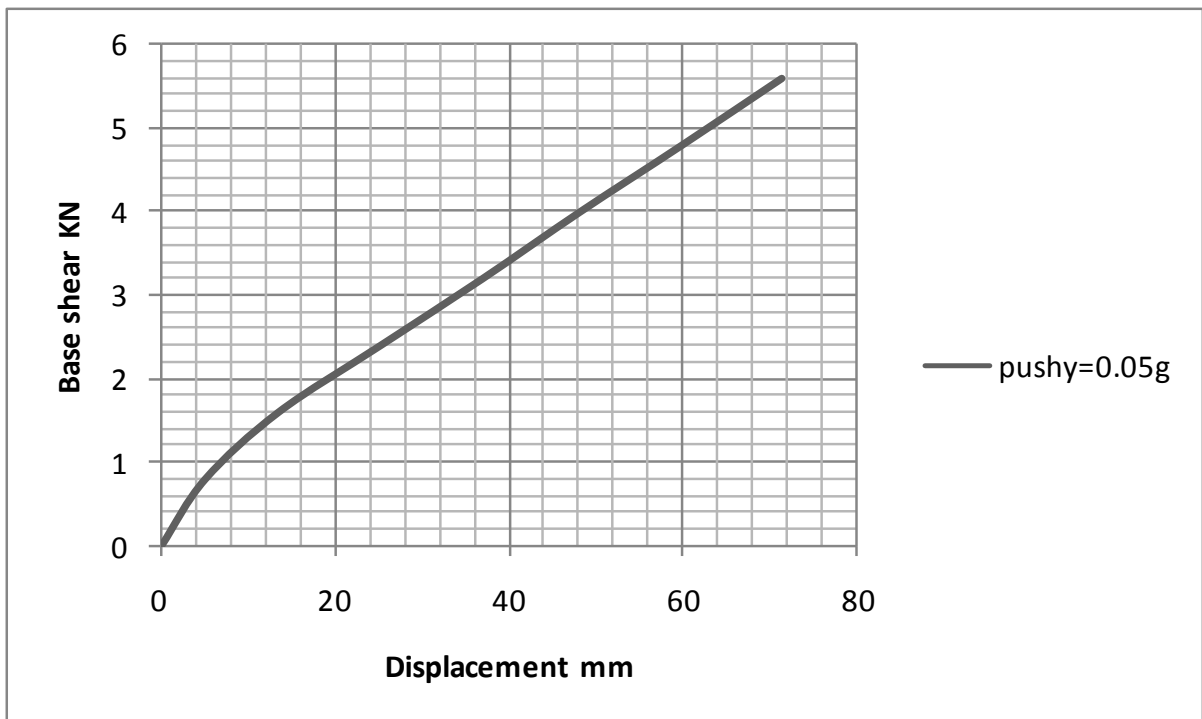


b)

Figure 5.15. Pushover curves of eight storey RC linear building with shear wall a) In X-direction (0.05g) b) In X-direction (0.1g)

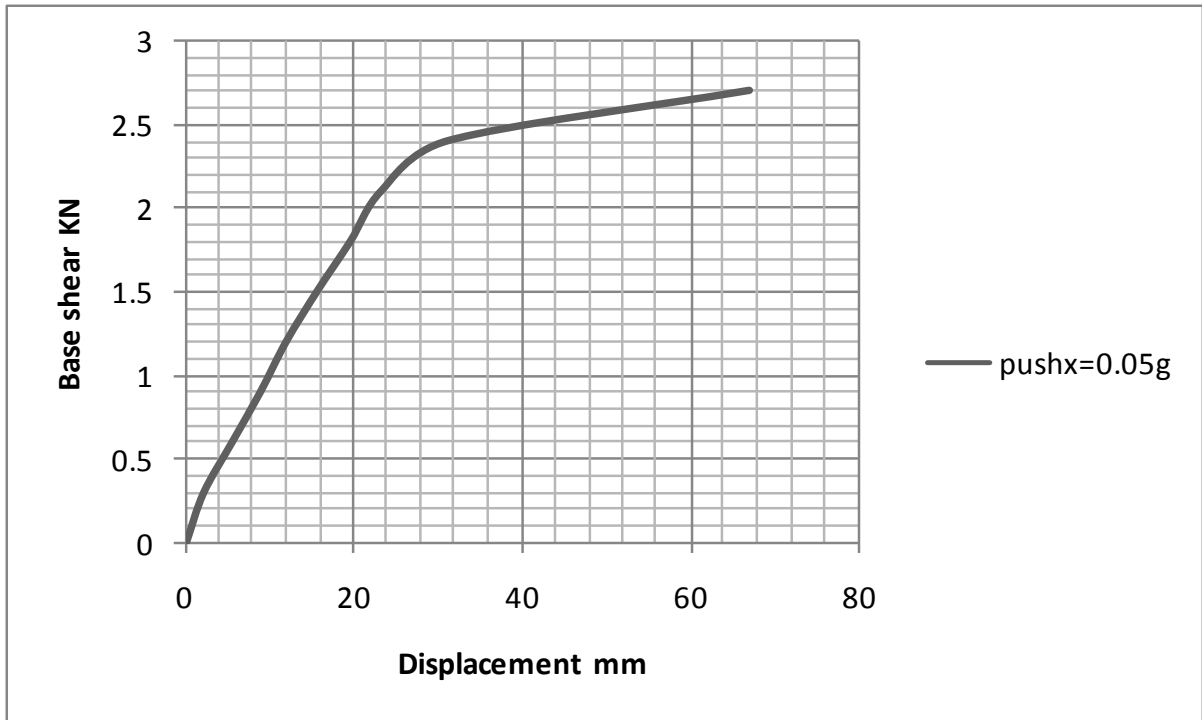


a)

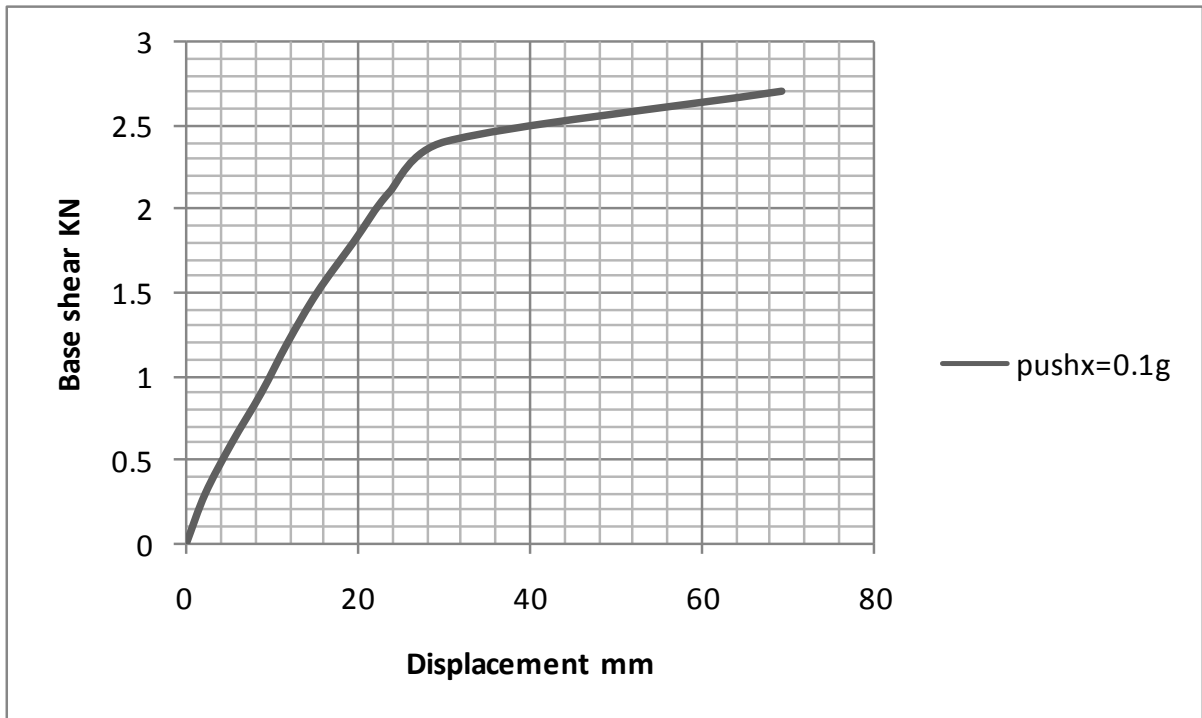


b)

Figure 5.16. Pushover curves of eight storey RC linear building with shear wall a) In Y-direction (0.05g) b) In Y-direction (0.1g)

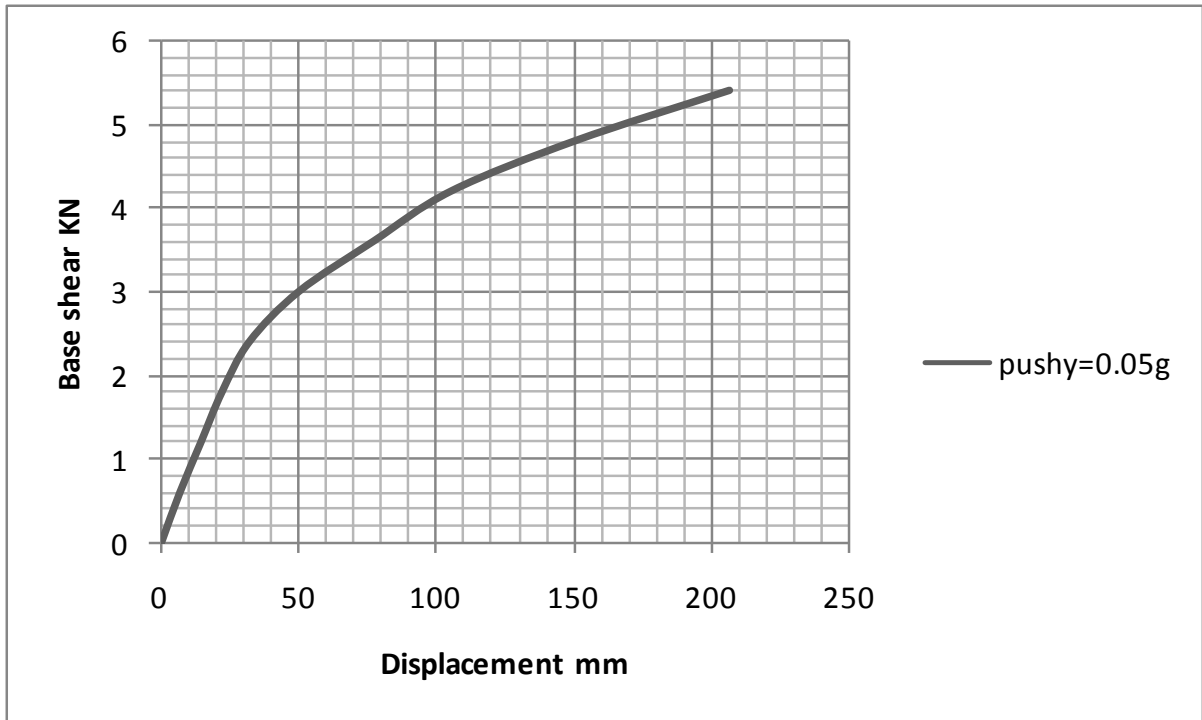


a)

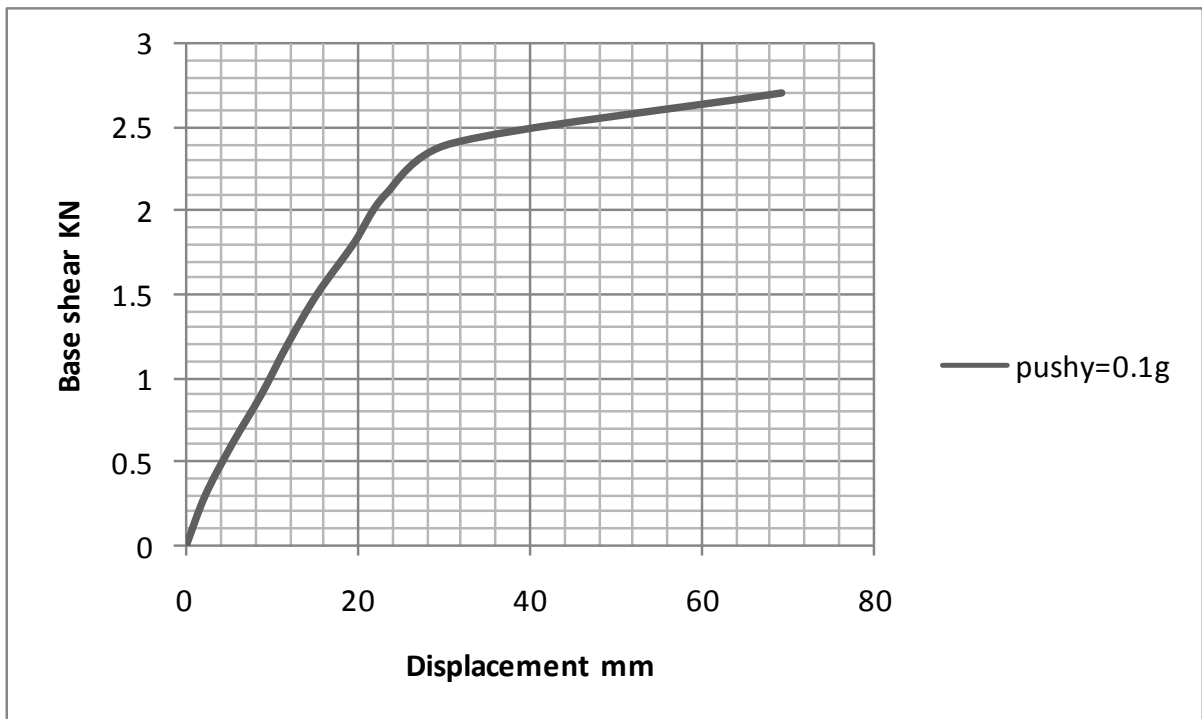


b)

Figure 5.17. Pushover curves of eight storey RC L-shaped or nonlinear building without shear wall a) In X-direction (0.05g) b) In X-direction (0.1g)

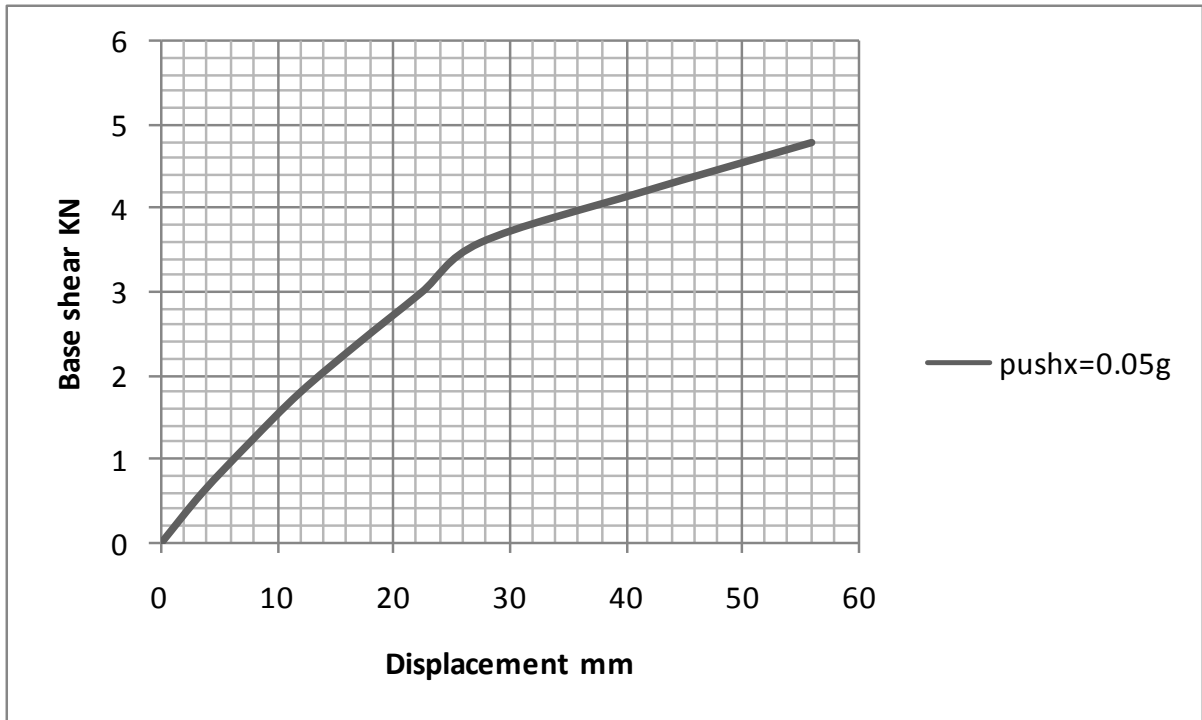


a)

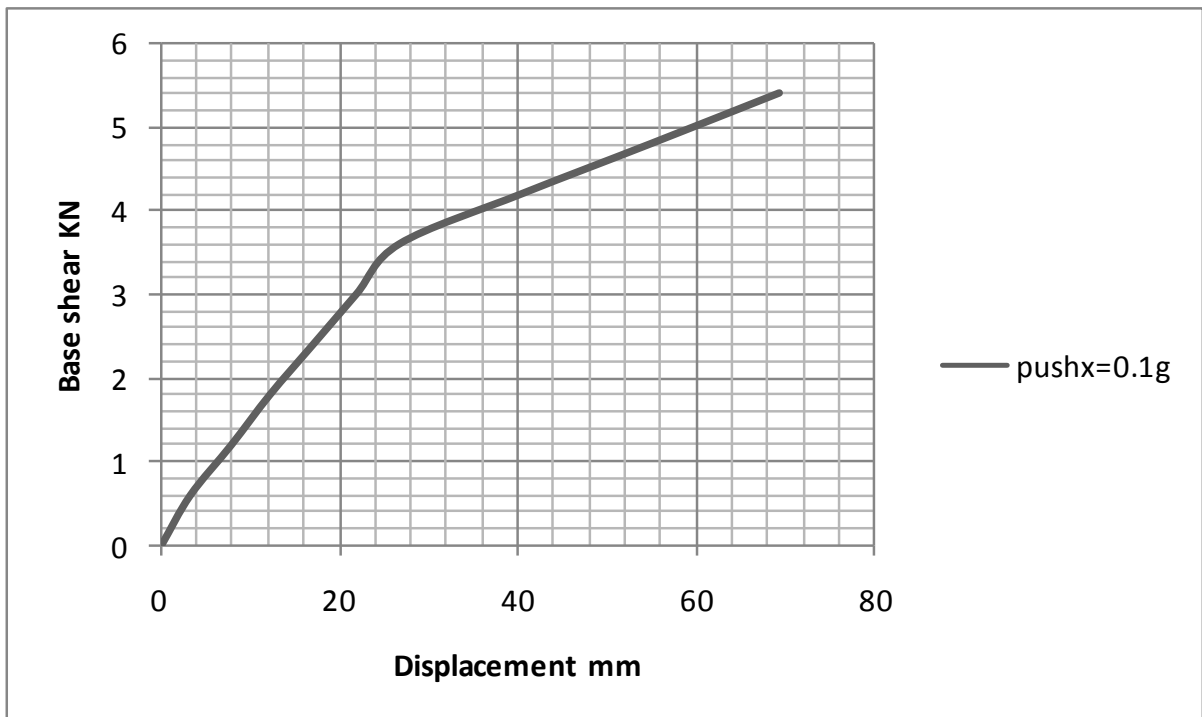


b)

Figure 5.18. Pushover curves of eight storey RC L-shaped building without shear wall a) In Y-direction (0.05g) b) In Y-direction (0.1g)

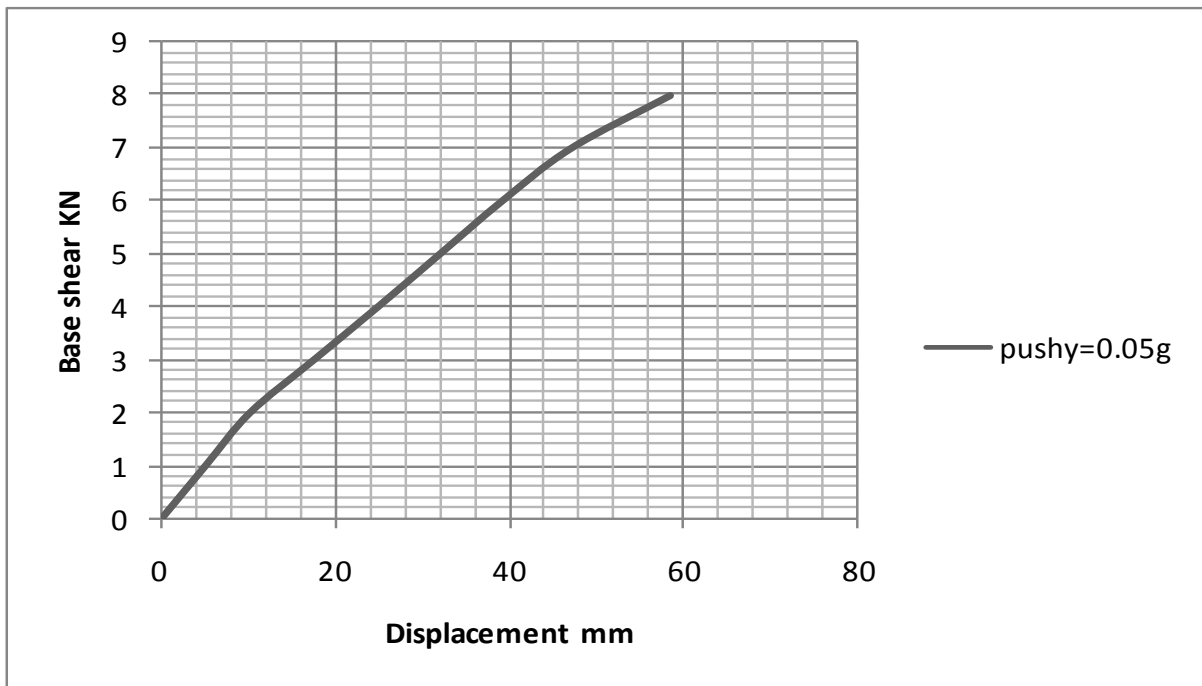


a)

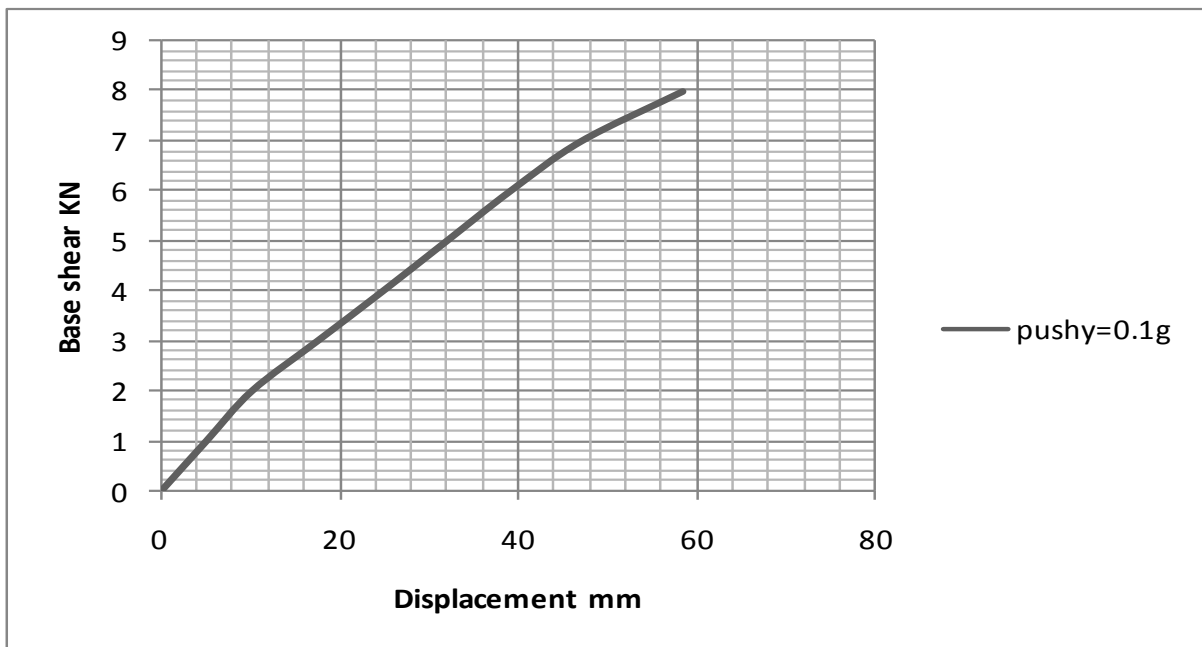


b)

Figure 5.19. Pushover curves of eight storey RC L-shaped building with shear wall



a)



b)

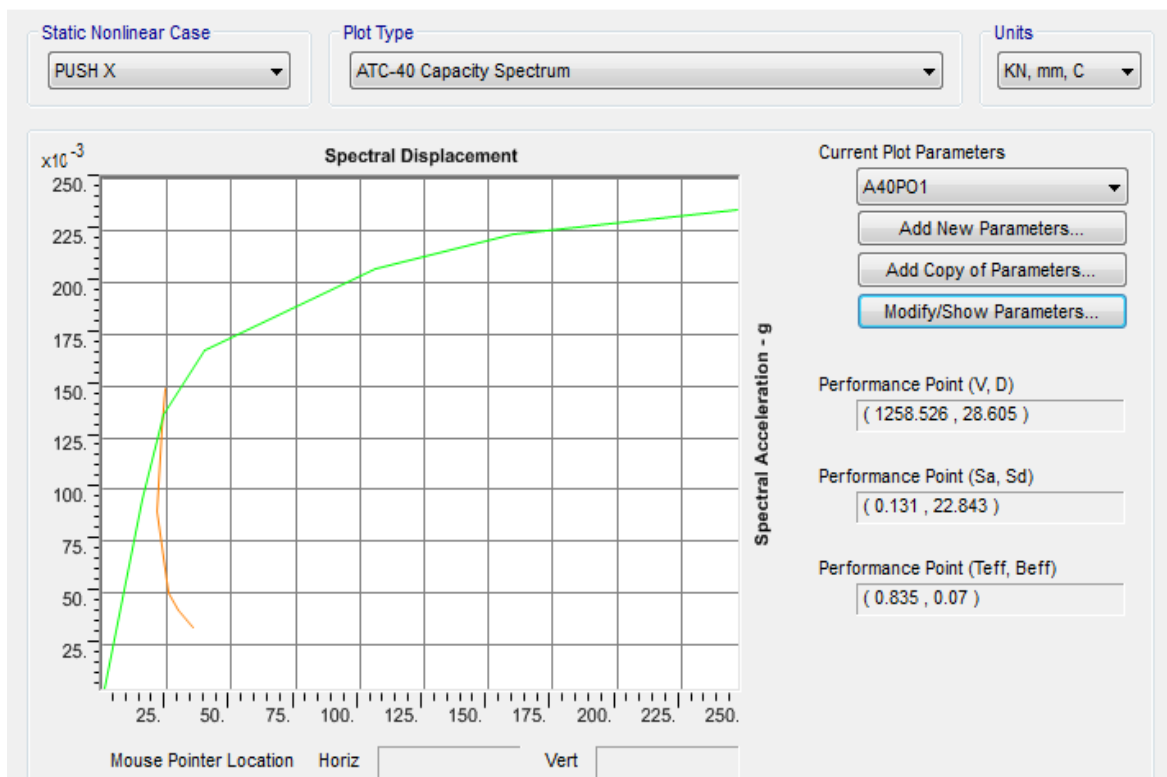
Figure 5.20. Pushover curves of eight storey RC L-shaped building with shear wall

a) In Y-direction (0.05g) b) In Y-direction (0.1g)

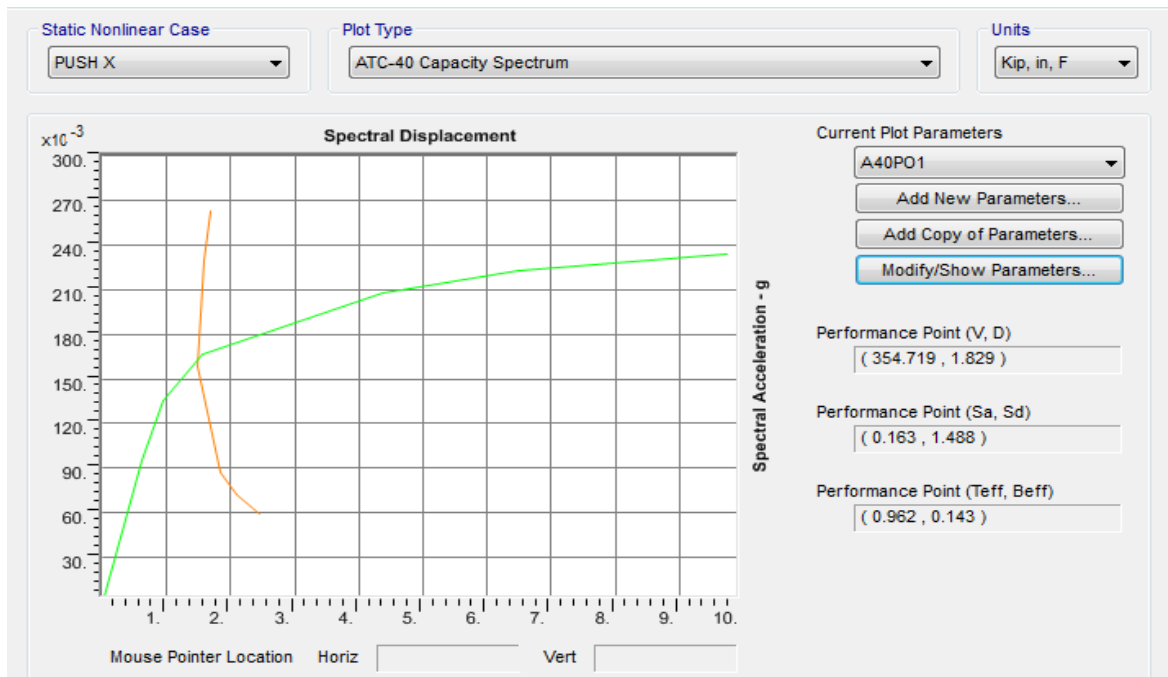
## 5.9 Performance points for the selected RC building

The performance point shown in figure below 5.21-5.28 respectively.

- The effect of the presence of shear wall in the old code/revised code over the performance point is the increase in the performance point (shear capacity) where shear wall exists relative to the structure without shear wall.
- In general the performance point of structures exhibits the increase in peak ground acceleration and the performance point accounting the revised code has increased the performance point of structure for both linear and non linear structures.
- The performance point slows the performance level or damage level of structures. It can be concluded that the presence of shear wall structures increases the performance capacity of structures.
- The design of structures for both linear and non linear using the revised code increase the performance capacity of structures.

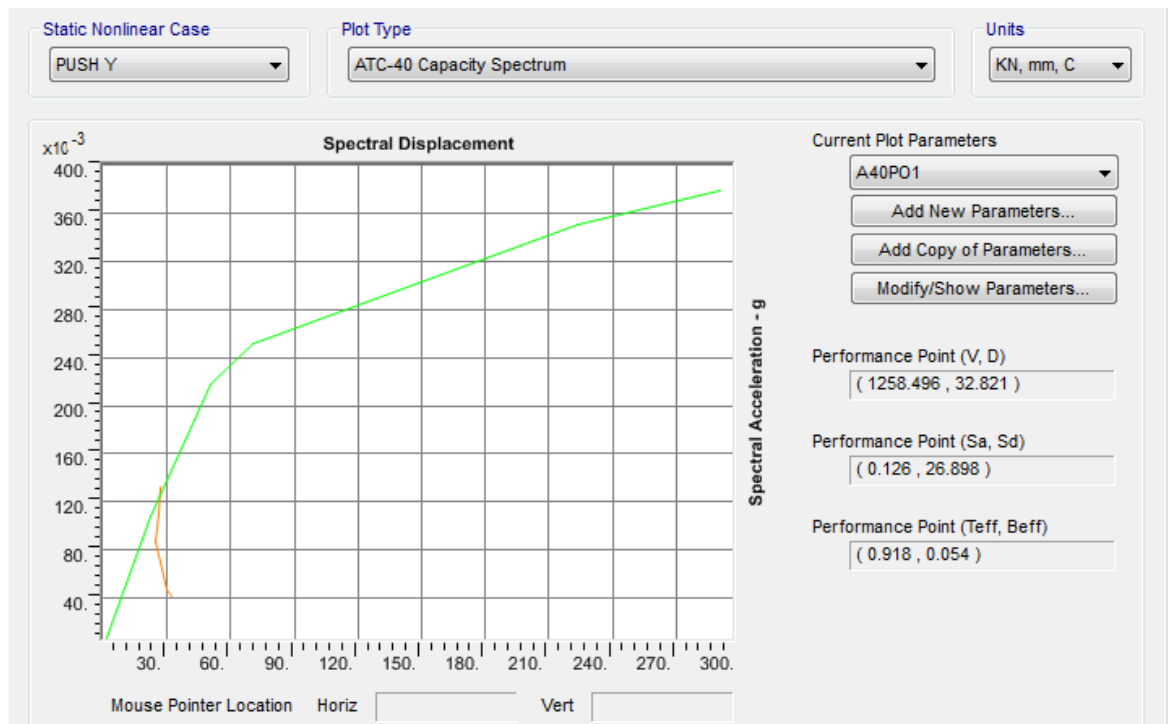


a)

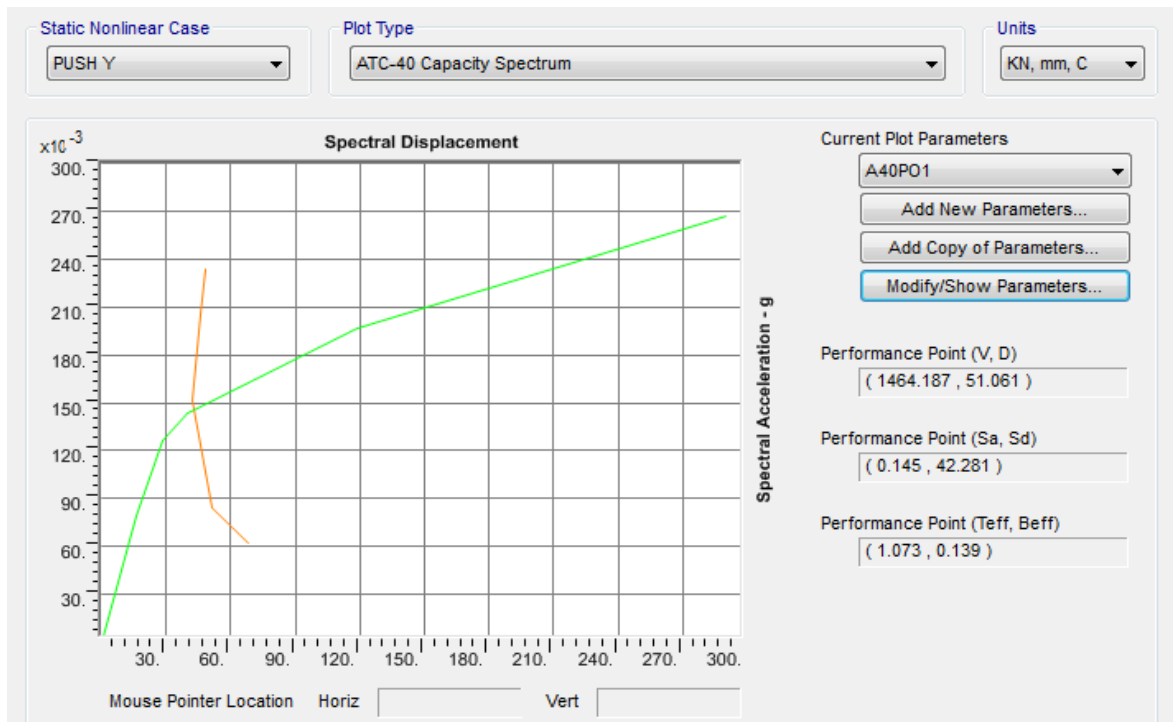


b)

Figure 5.21. performance points for eight storey linear RC building without shear wall a) In X-direction(0.05g) and b)In X-direction(0.1g)

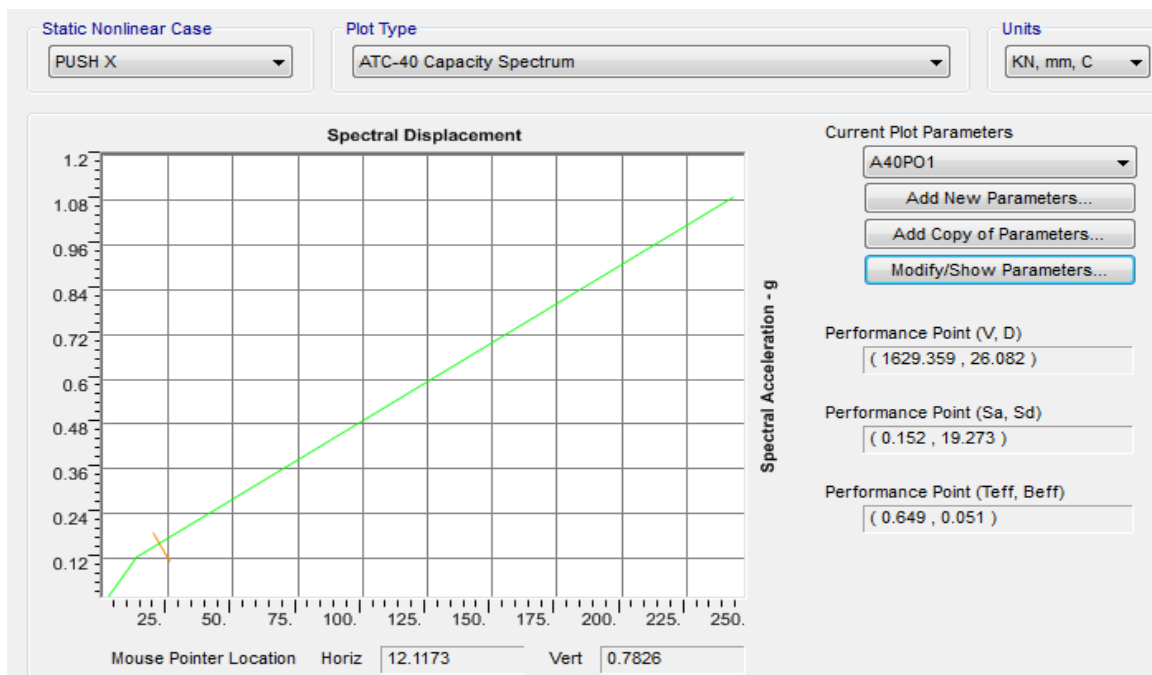


a)

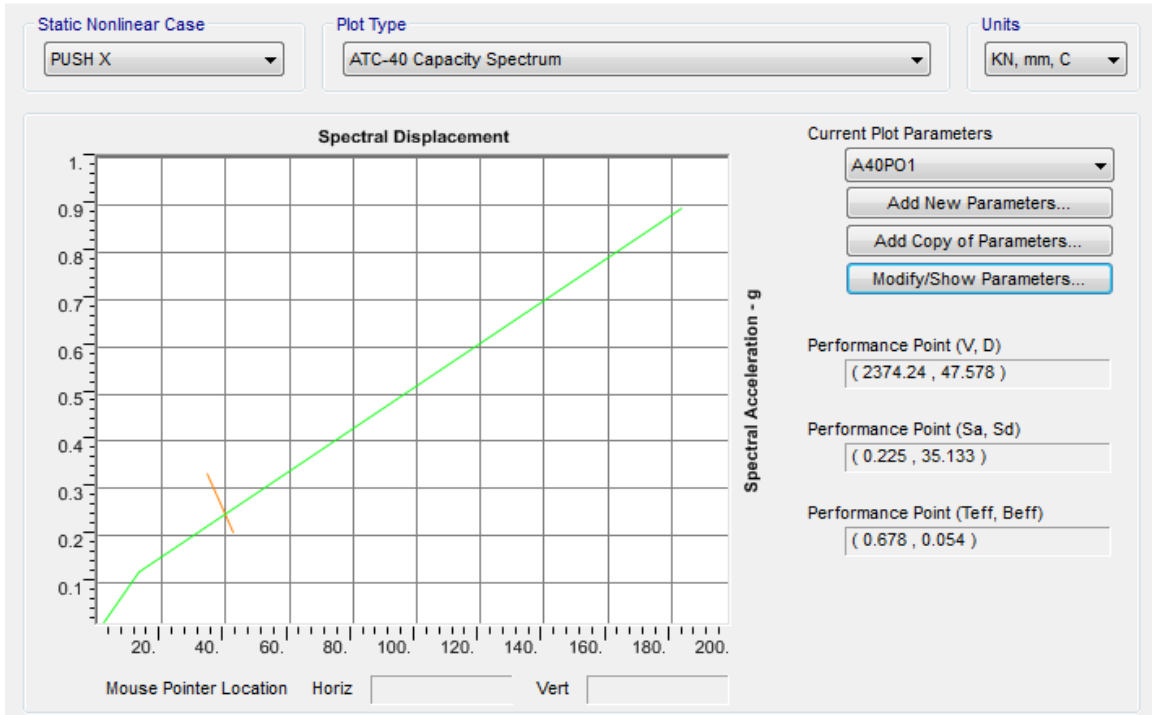


b)

Figure 5.22. Performance points for eight storey linear RC building without shear wall a) In Y-direction(0.05g) b) In Y-direction(0.1g)

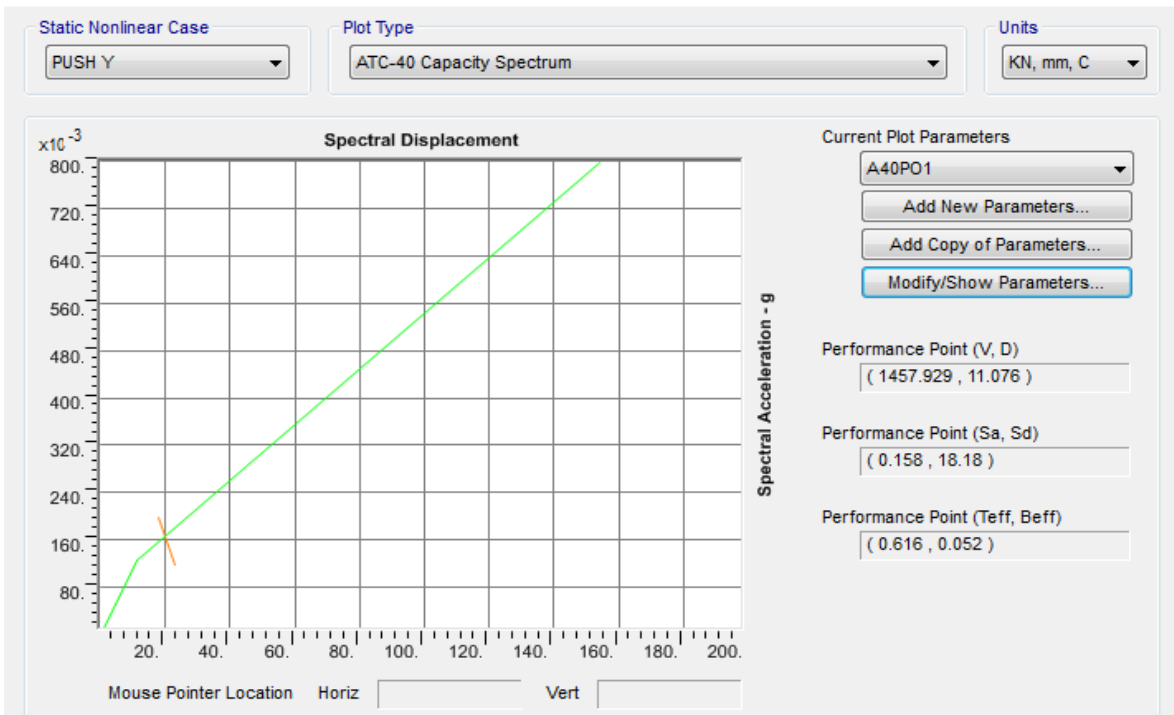


a)

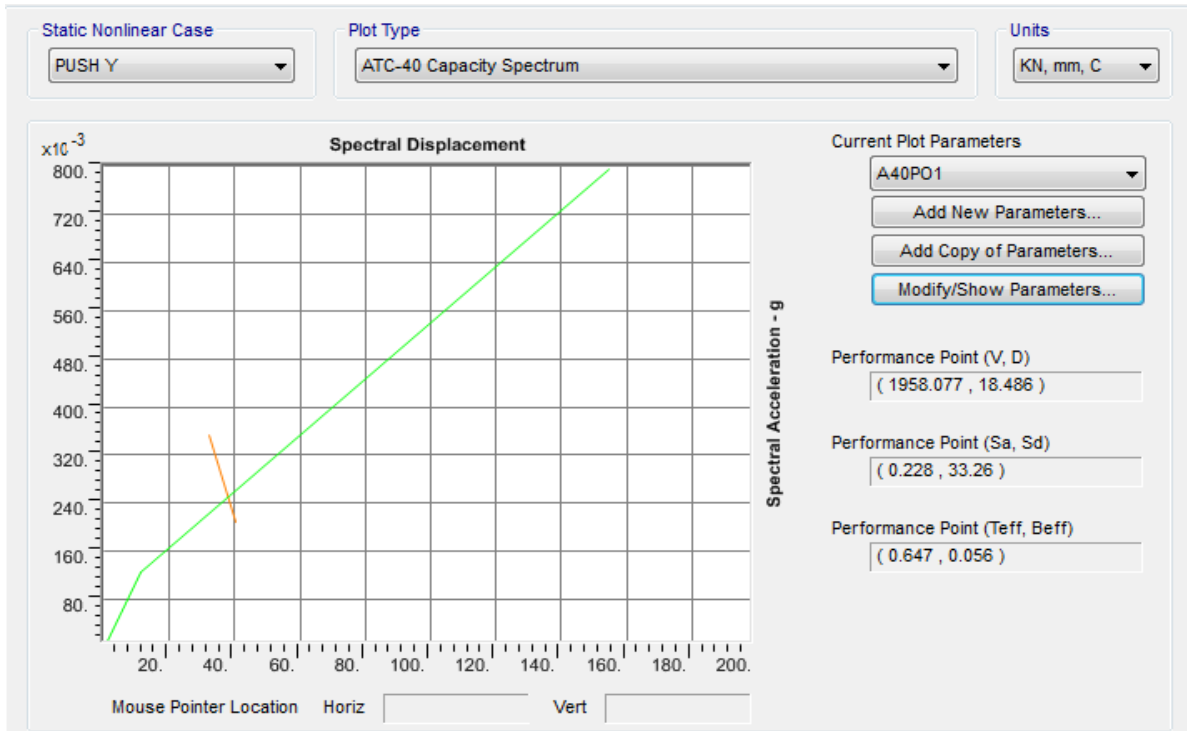


b)

Figure 5.23. Performance points for eight storey linear RC building with shear wall a) In X-direction(0.05g) b) In X-direction(0.1g)

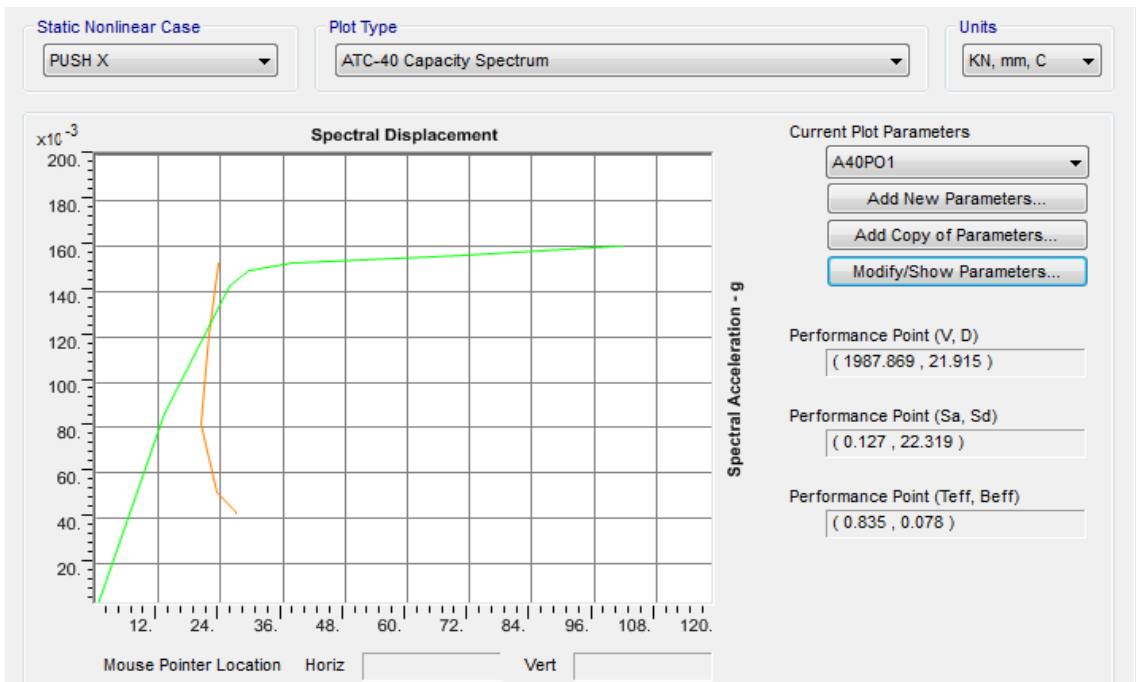


a)

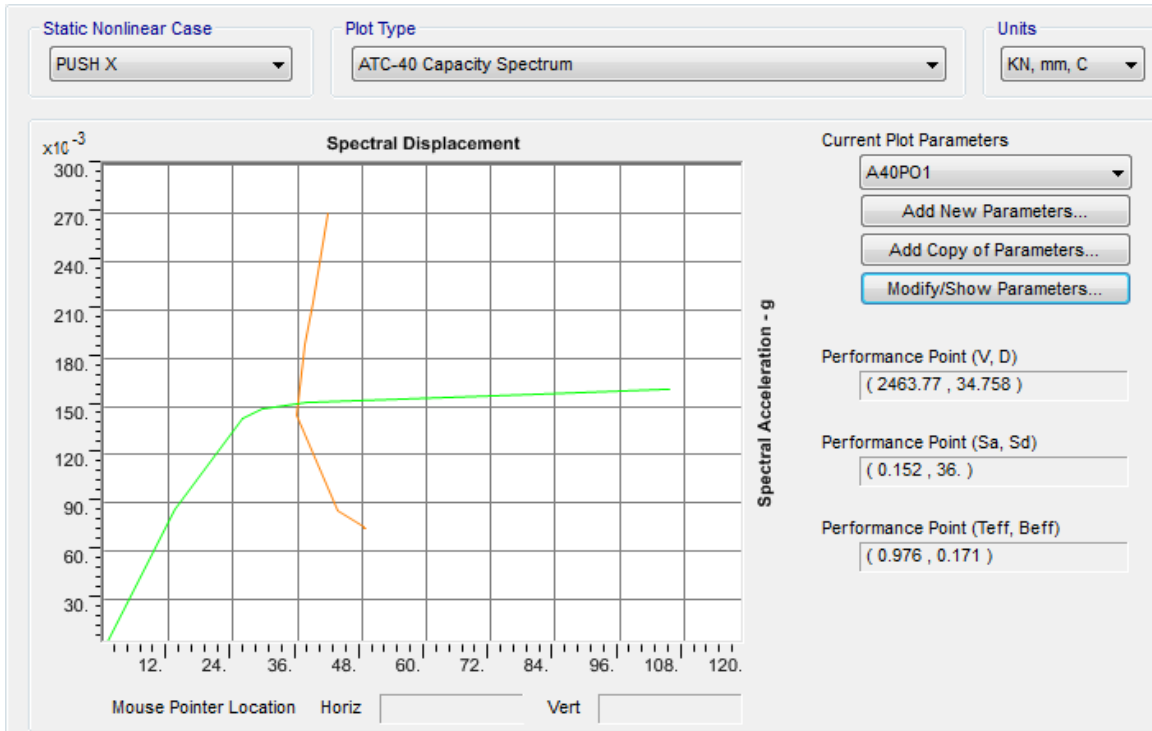


b)

Figure 5.24. Performance points for eight storey linear RC building with shear wall a) In Y-direction(0.05g) b) In Y-direction(0.1g)

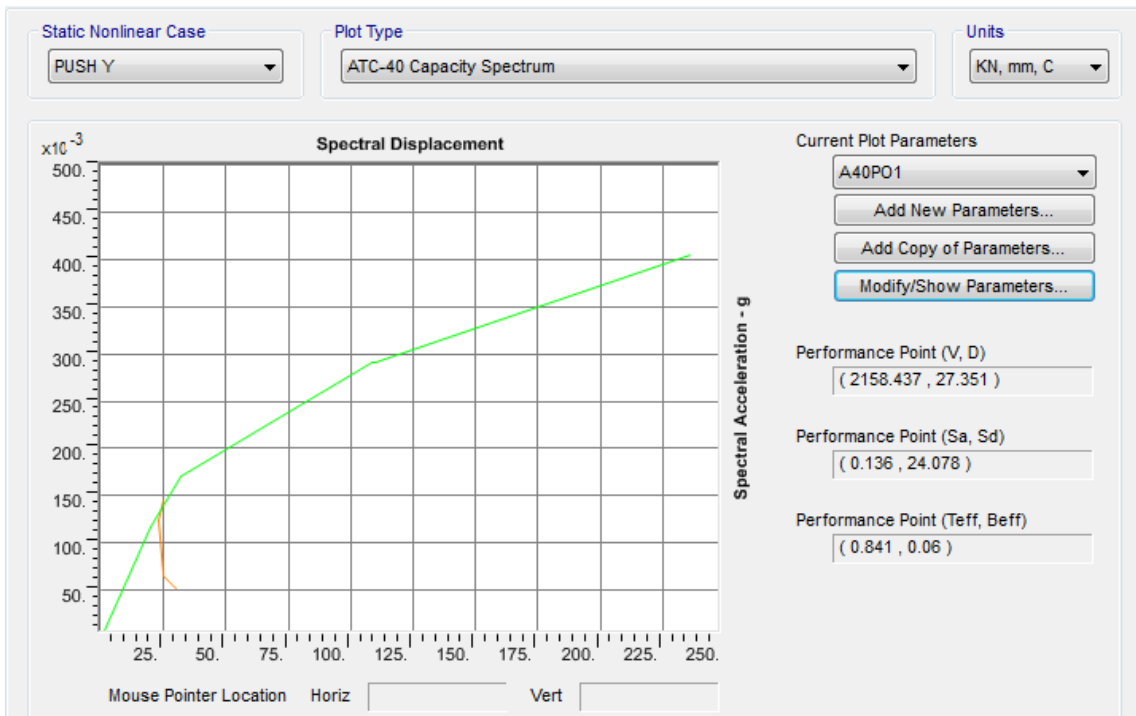


a)

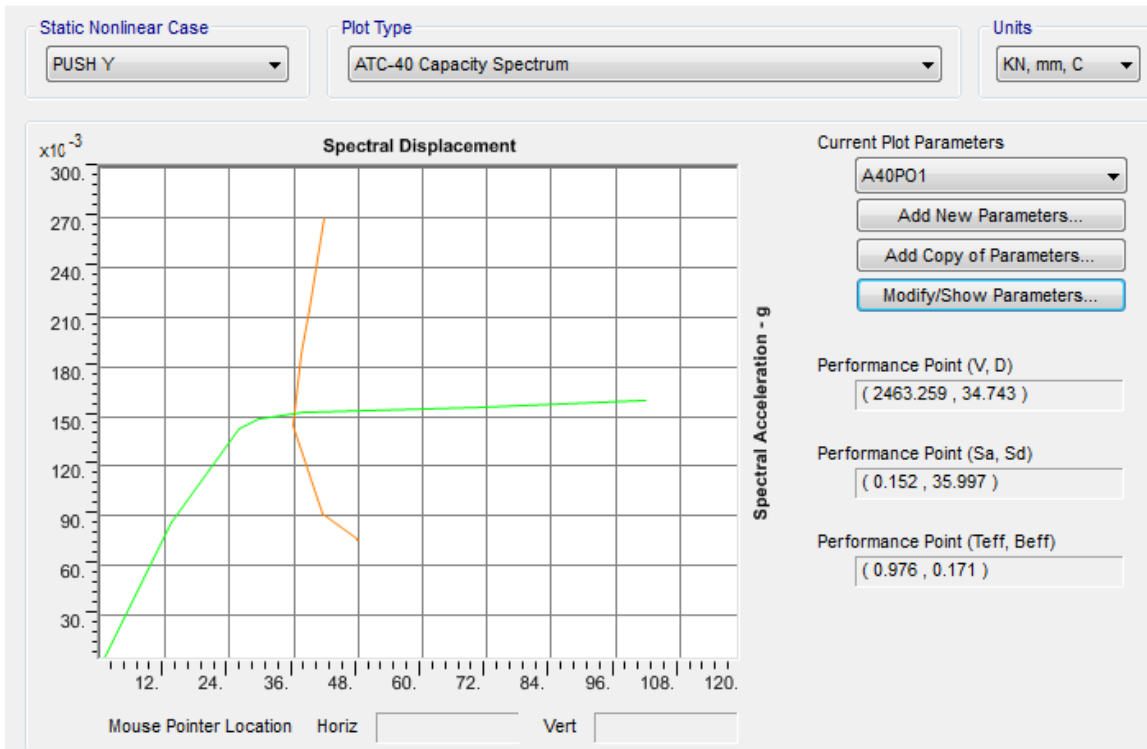


b)

Figure 5.25. Performance points for L-shaped or nonlinear eight storey RC building without shear wall a) In X-direction(0.05g) b) In X-direction(0.1g)

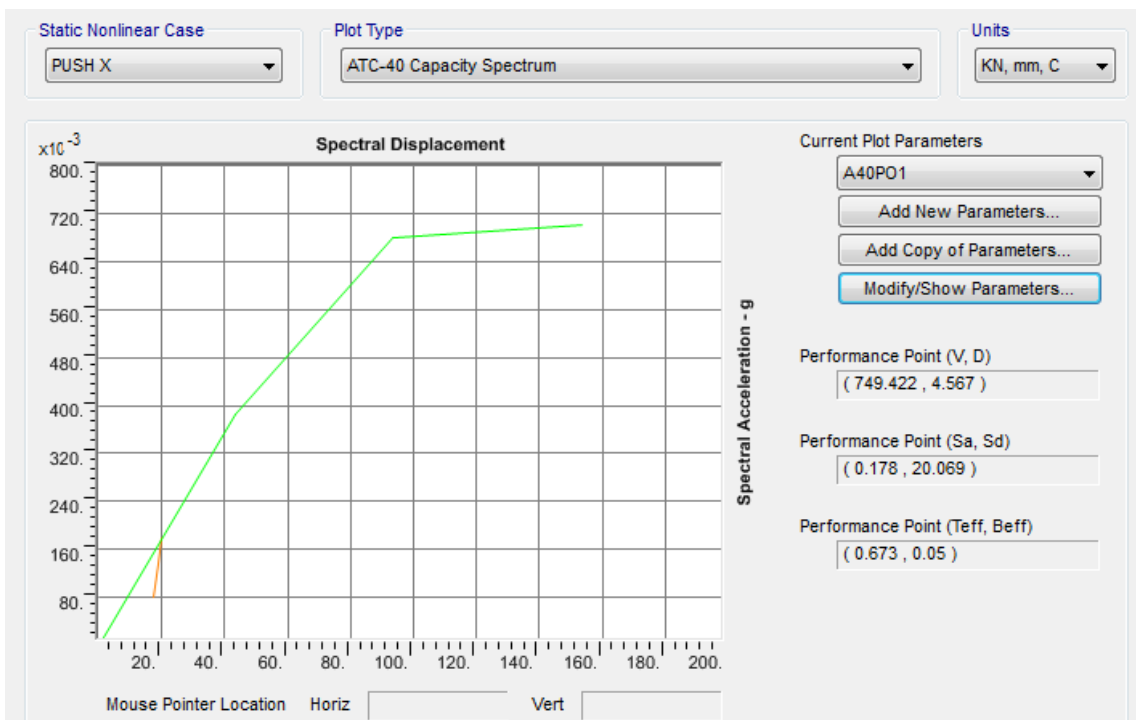


a)

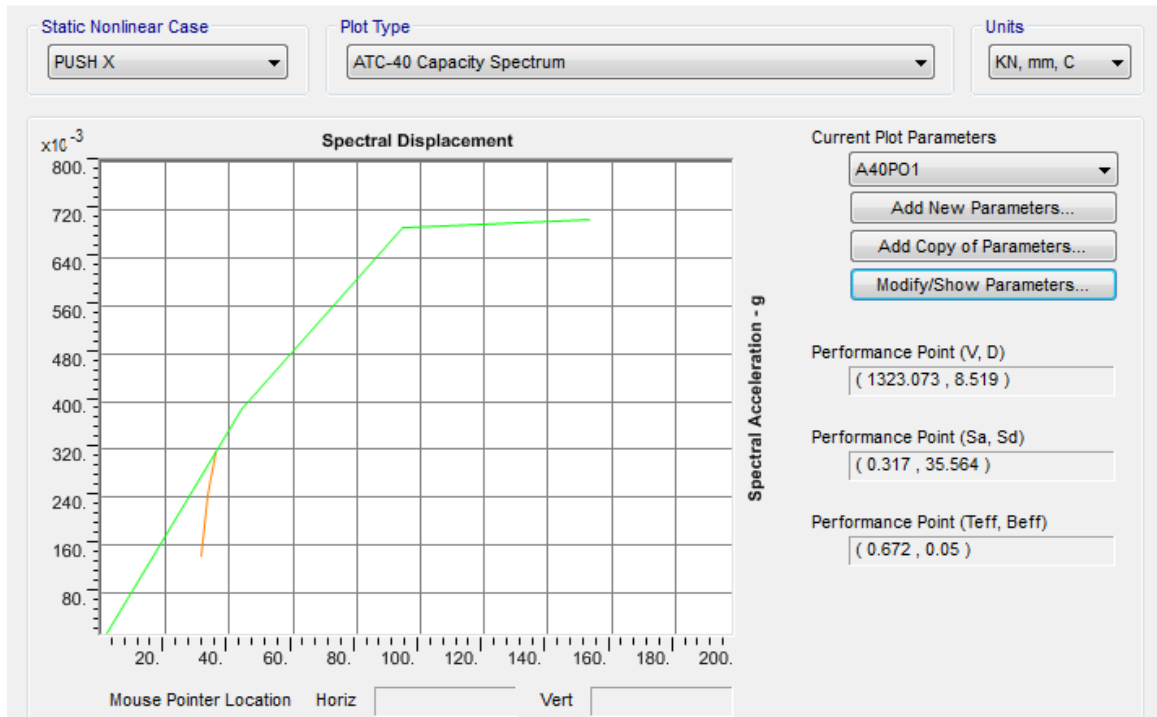


b)

Figure 5.26. Performance points for L-shaped or nonlinear eight storey RC building without shear wall a) In Y-direction (0.05g) b) In Y-direction (0.1g)

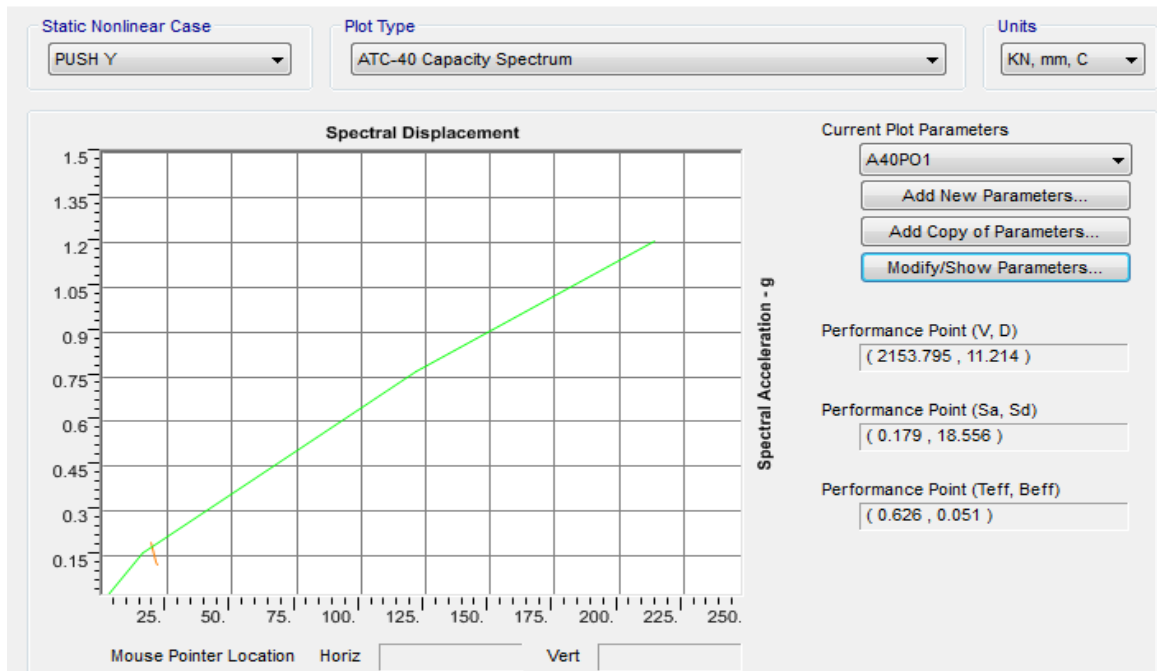


a)

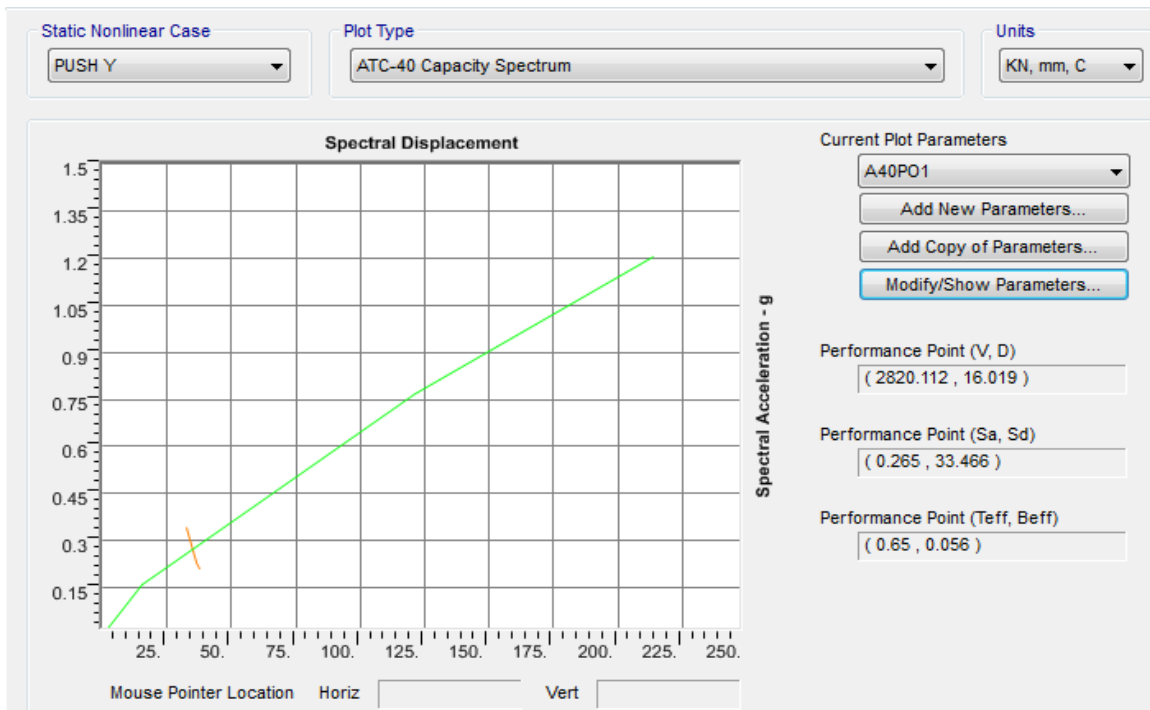


b)

Figure 5.27. Performance points for L-shaped or nonlinear eight storey RC building with shear wall a) In X-direction (0.05g) b) In X-direction (0.1g)



a)



b)

Figure 5.28. Performance points for L-shaped or nonlinear eight storey RC building with shear wall a) In Y-direction (0.05g) b) In Y-direction (0.1g)

Table 5.12 Summary of performance point for linear building structures

Direction	Code	With and Without Shear wall	Building Types	Performance Points	
				Base Shear(KN)	Displacement(mm)
X-Direction	Previous	Without	Regular	1258.53	28.61
Y-Direction	Previous	Without	Regular	1258.51	32.82
X-Direction	Previous	With	Regular	1629.36	26.08
Y-Direction	Previous	With	Regular	1457.93	11.1
X-Direction	Revised	Without	Regular	1577.87	46.45
Y-Direction	Revised	Without	Regular	2008.33	57.46
X-Direction	Revised	With	Regular	2374.24	47.58
Y-Direction	Revised	With	Regular	1958.08	18.49

previous and Revised represents the value of peak ground acceleration

Table 5.13 Summary of performance point for Nonlinear building structures

Direction	Code	With and Without	Building Types	Performance Points	
				Base Shear(KN)	Displacement(mm)
		<b>Shear wall</b>			
X-Direction	Previous	Without	Irregular	1987.87	21.92
Y-Direction	Previous	Without	Irregular	2158.44	34.76
X-Direction	Previous	With	Irregular	749.4	4.57
Y-Direction	Previous	With	Irregular	2151.16	11.2
X-Direction	Revised	Without	Irregular	2463.77	34.76
Y-Direction	Revised	Without	Irregular	2463.77	34.76
X-Direction	Revised	With	Irregular	1323.07	8.52
Y-Direction	Revised	With	Irregular	2805.32	15.95
previous and Revised represents the value of peak ground acelaration					

### 5.10 Plastic hinge distribution

Even though, asymmetrical structures are subjected to pair of actions at a time when subjected to horizontal earthquake forces, pushover analysis is carried out only in the direction of larger displacement. Generally, it has been found that the investigated structures in both X and Y direction subjected the appropriate load combination have remained within the immediate occupancy performance level. The formation of hinges at the last step of pushover analysis; for linear and L-shaped eight storey RC building with and without including code variation (0.05g(old) and 0.1g(new) Ethiopian building code) for both have been checked.

The performance point shown in the figure below 5.29-5.42 displays the performance level for linear building exhibits large number of LS ,some CP and collapse point but the performance level of non-linear building exhibits some number of life safety and few number of collapse but not damage point at all.

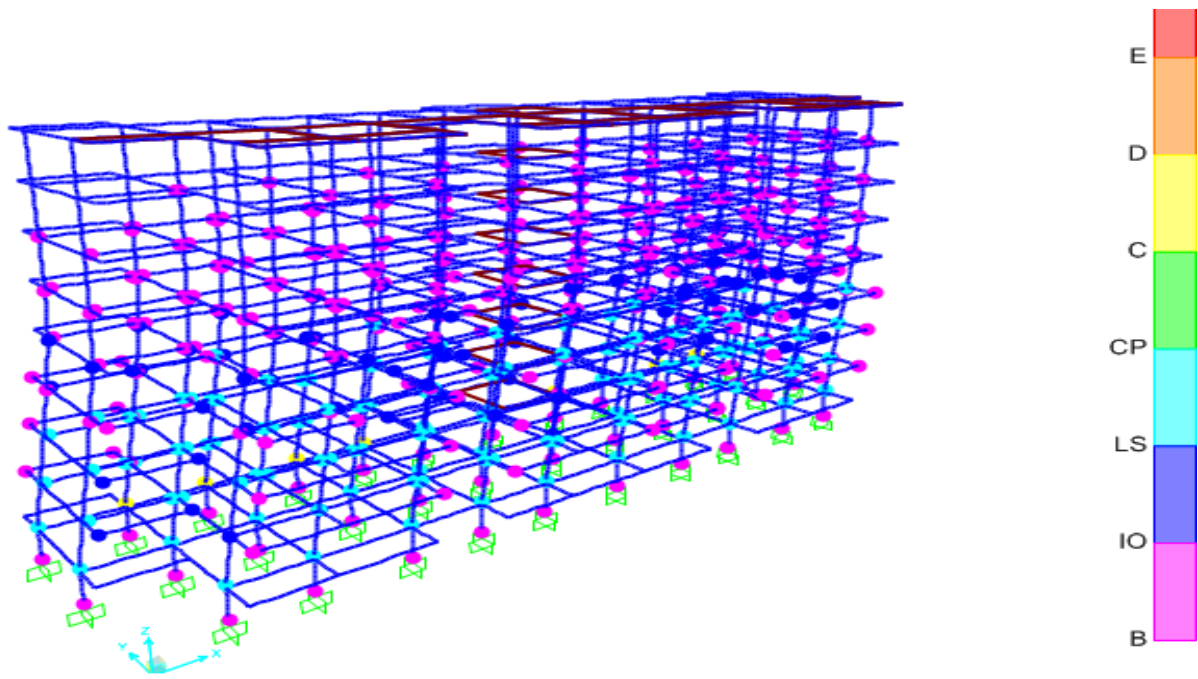


Figure 5.29. Plastic hinge distribution in eight storey linear RC building without shear in X-direction (0.05g).

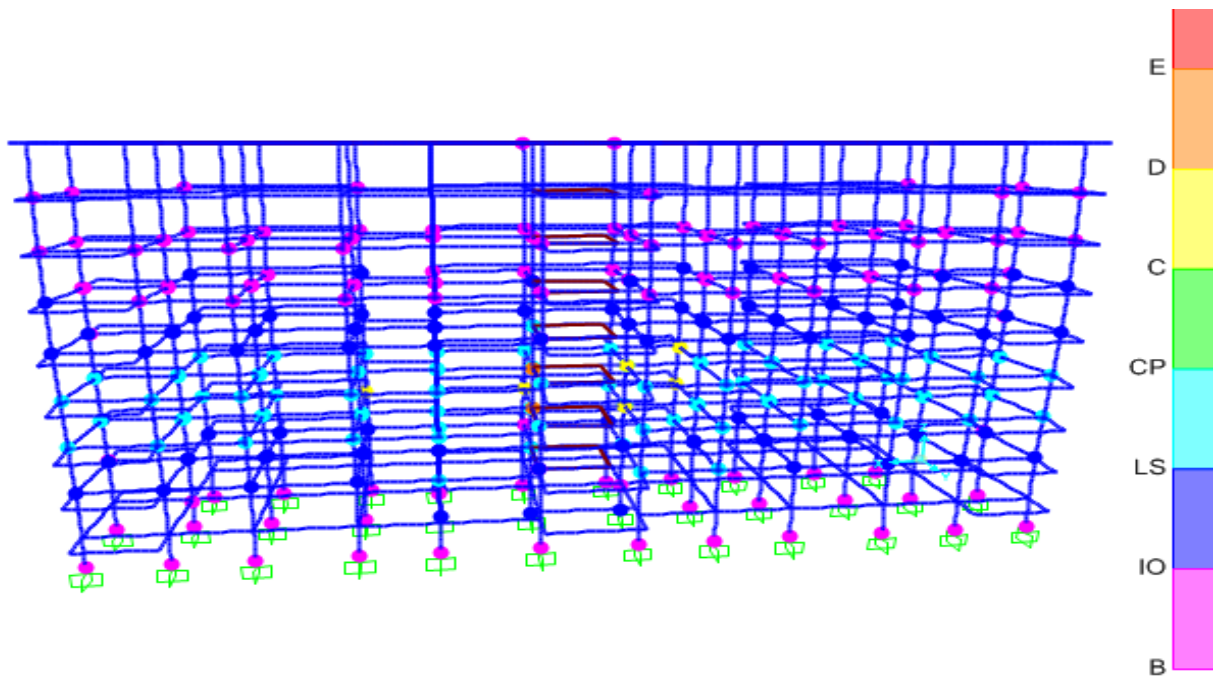


Figure 5.30. Plastic hinge distribution in eight storey linear RC building in Y-direction(0.05g)

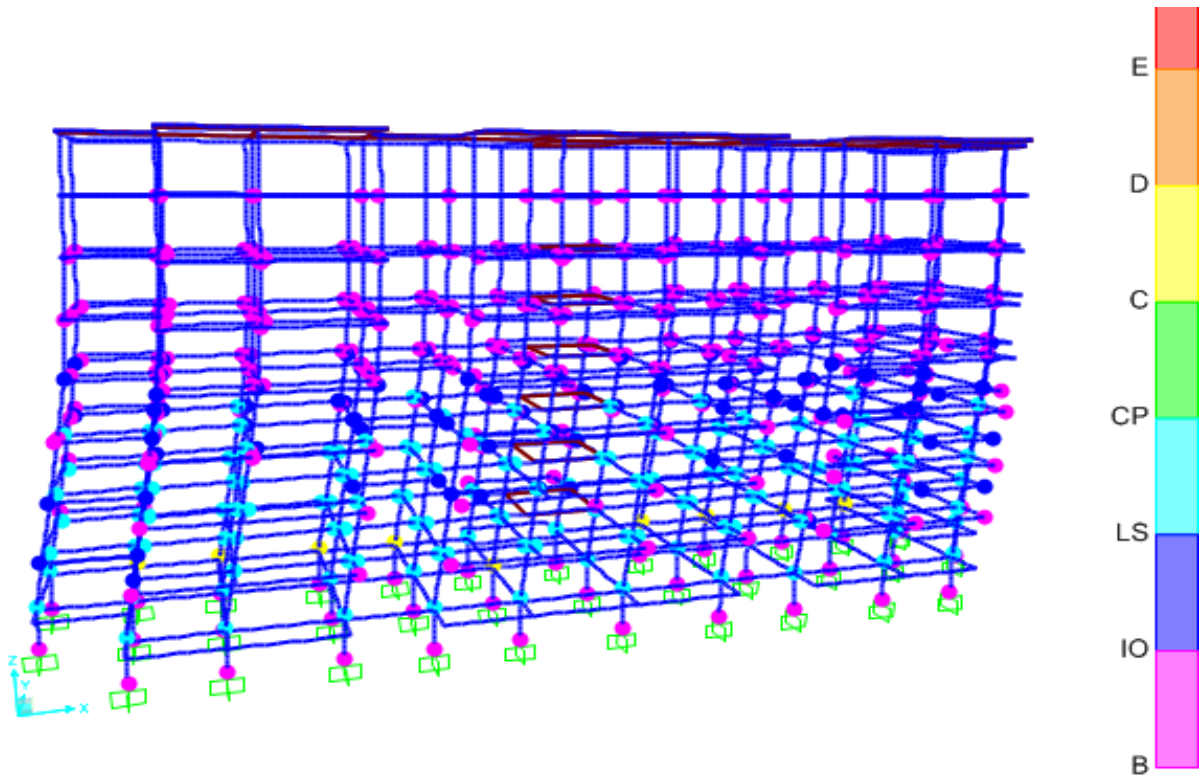


Figure 5.31. Plastic hinge distribution in eight storey linear RC building without shear wall in X-direction (0.1g)

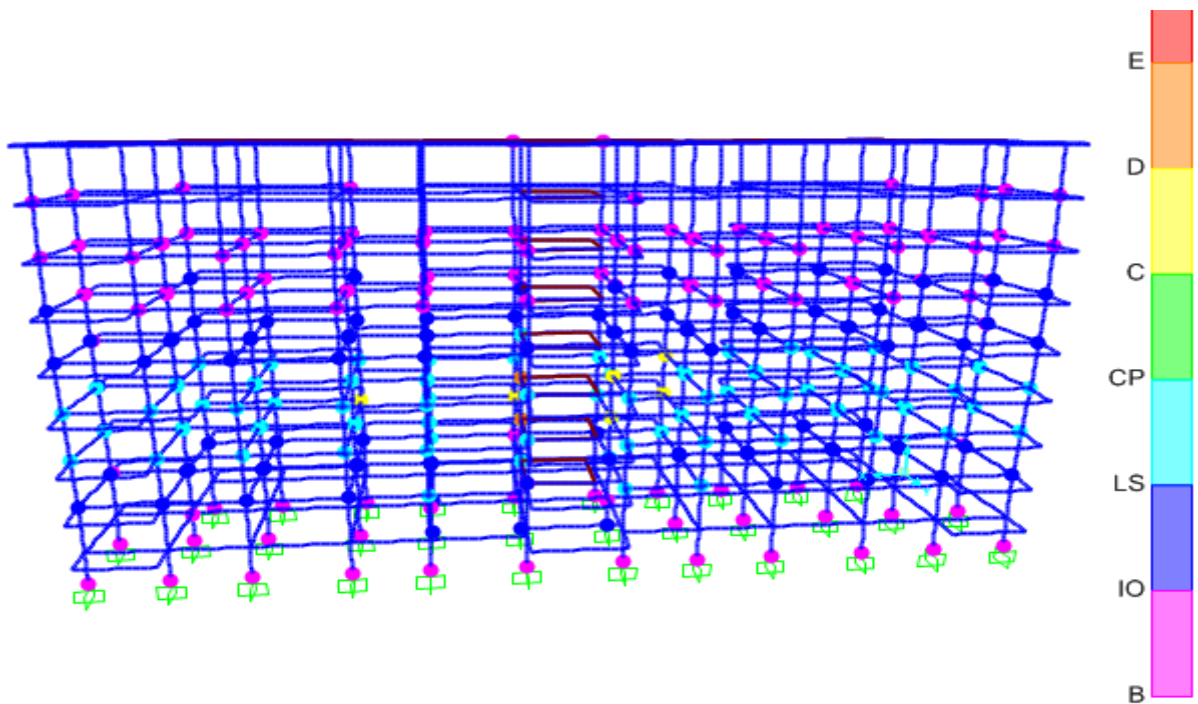


Figure 5.32. Plastic hinge distribution in eight storey linear RC building without shear wall in Y-direction (0.1g)

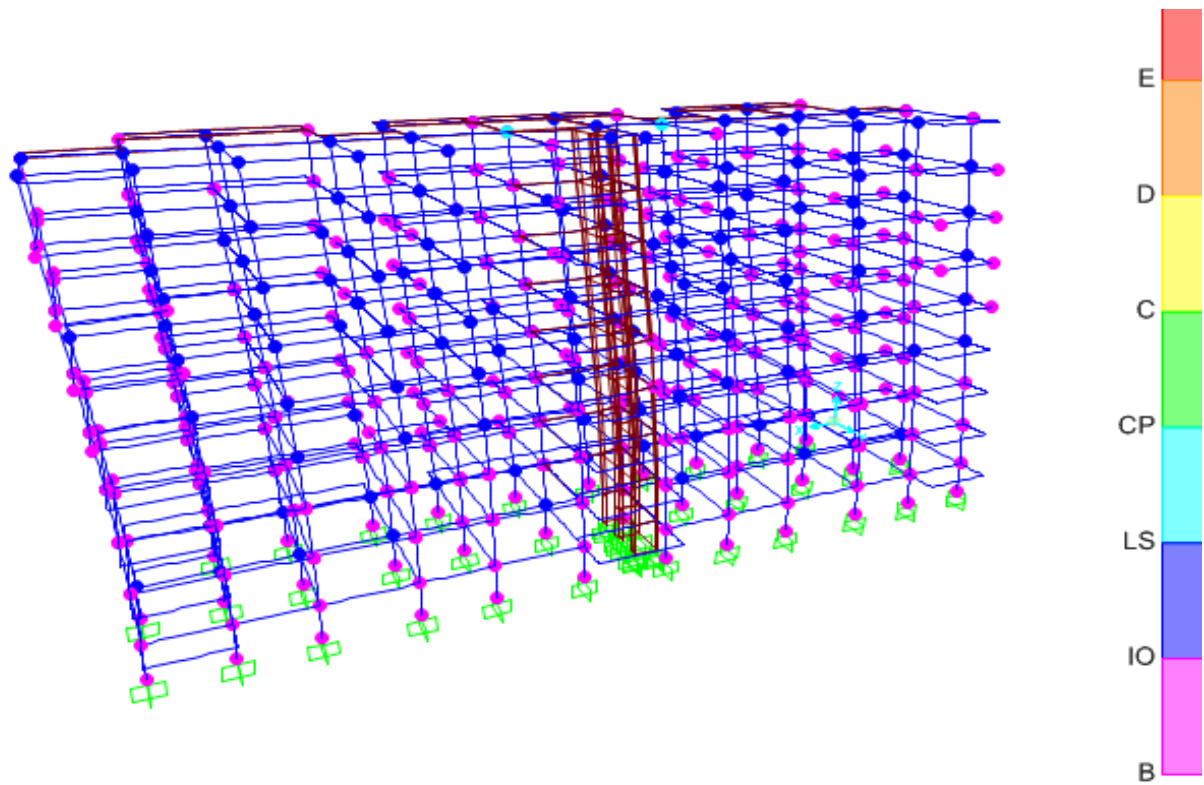


Figure 5.33. Plastic hinge distribution in eight storey linear RC building with shear wall in X-direction (0.05g)

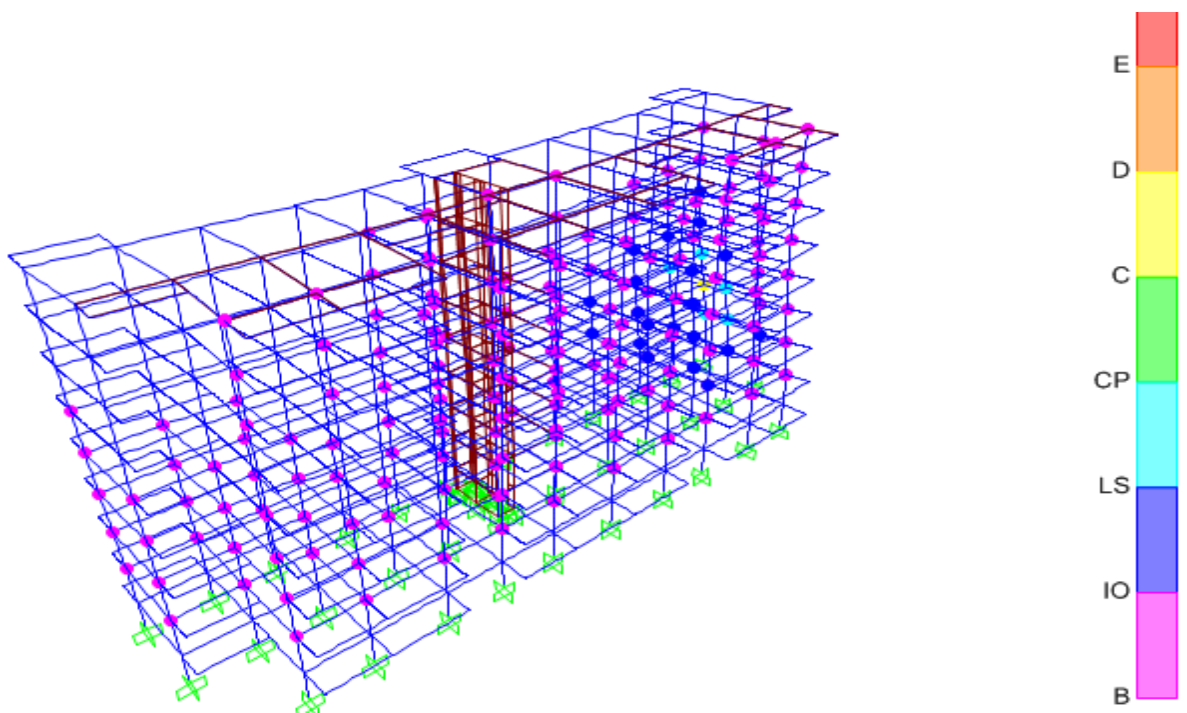


Figure 5.34. Plastic hinge distribution in eight storey linear RC building with shear wall in Y-direction (0.05g)

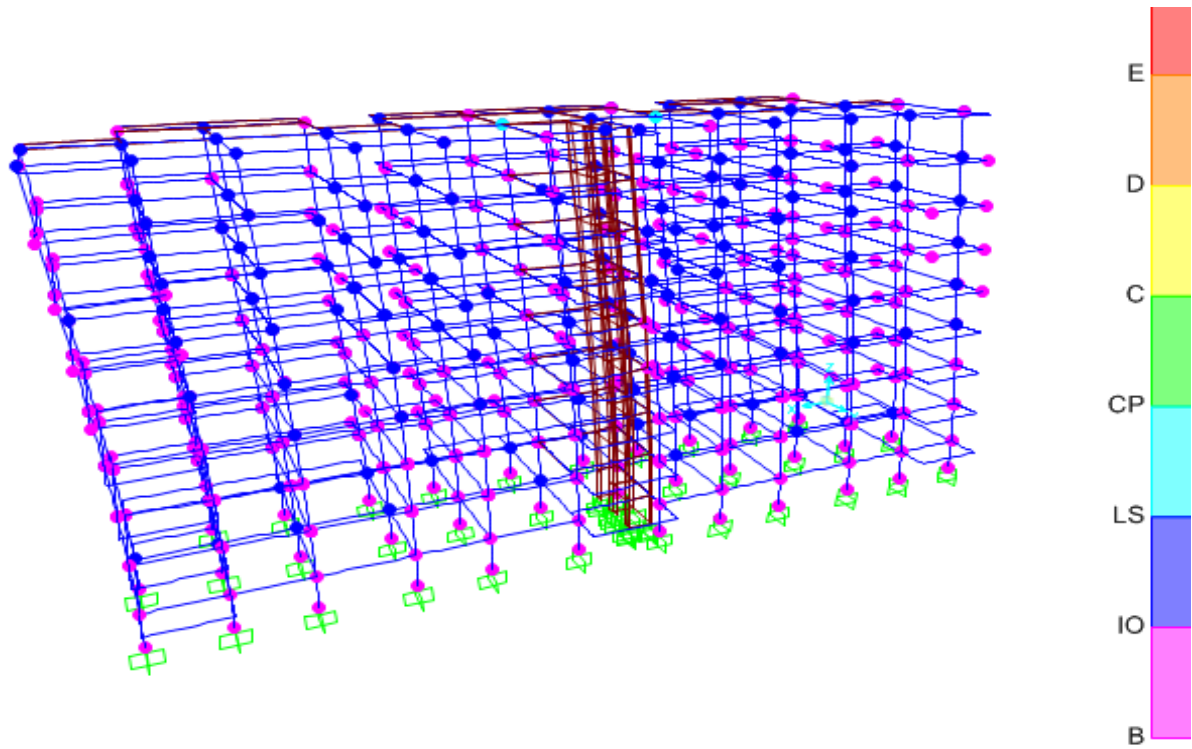


Figure 5.35. Plastic hinge distribution in eight storey linear RC building with shear wall in X-direction (0.1g)

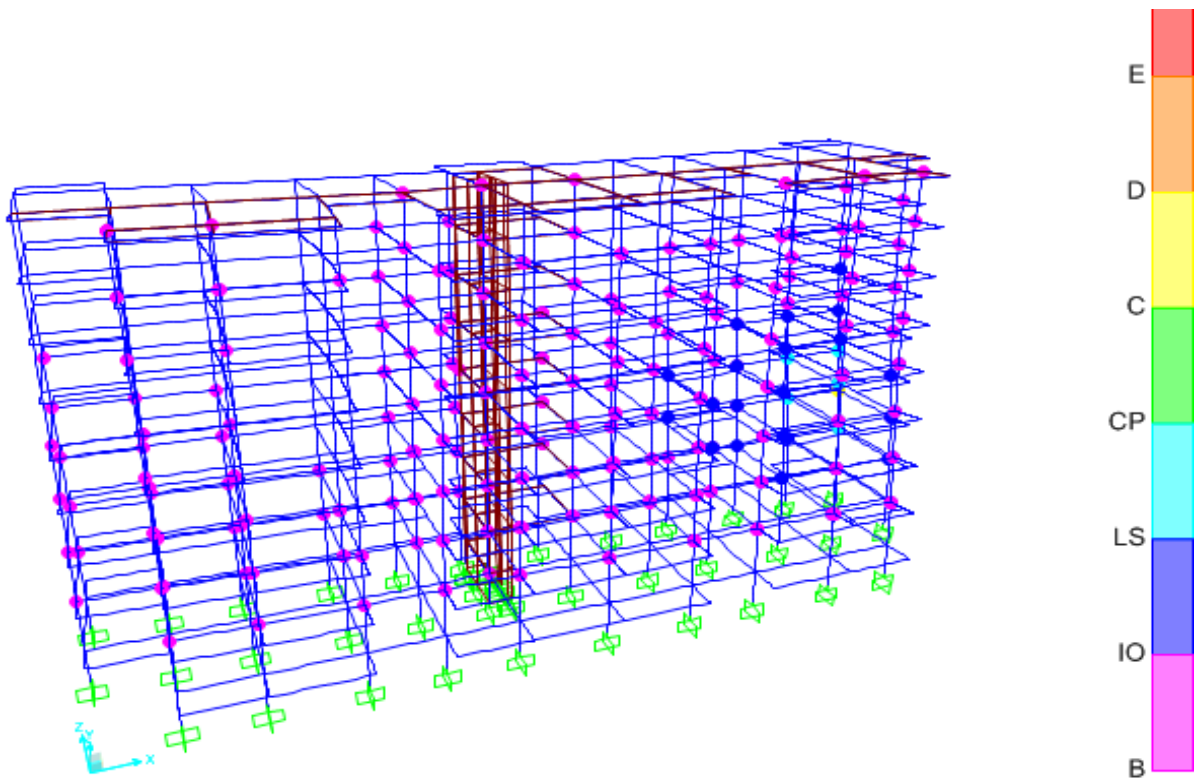


Figure 5.36. Plastic hinge distribution in eight storey linear RC building with shear wall in Y-direction (0.1g)

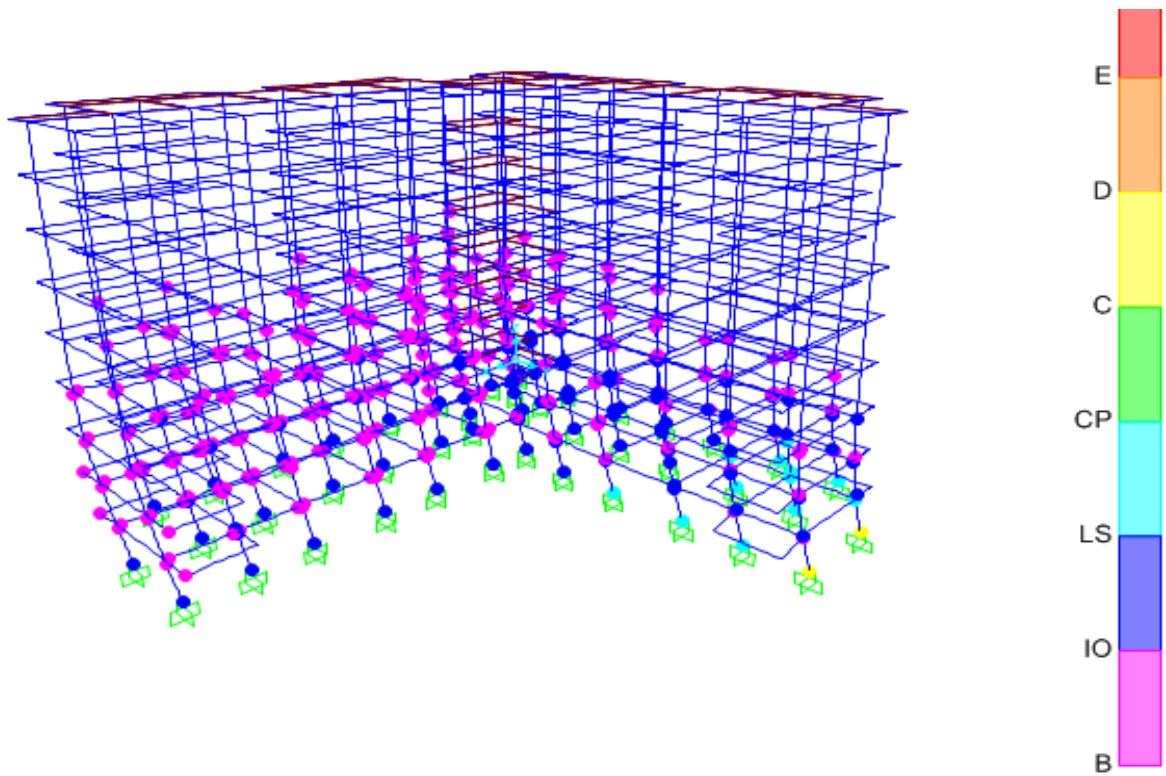


Figure 5.37. Plastic hinge distribution in eight storey L-shaped or nonlinear RC building without shear wall in X-direction (0.05g)

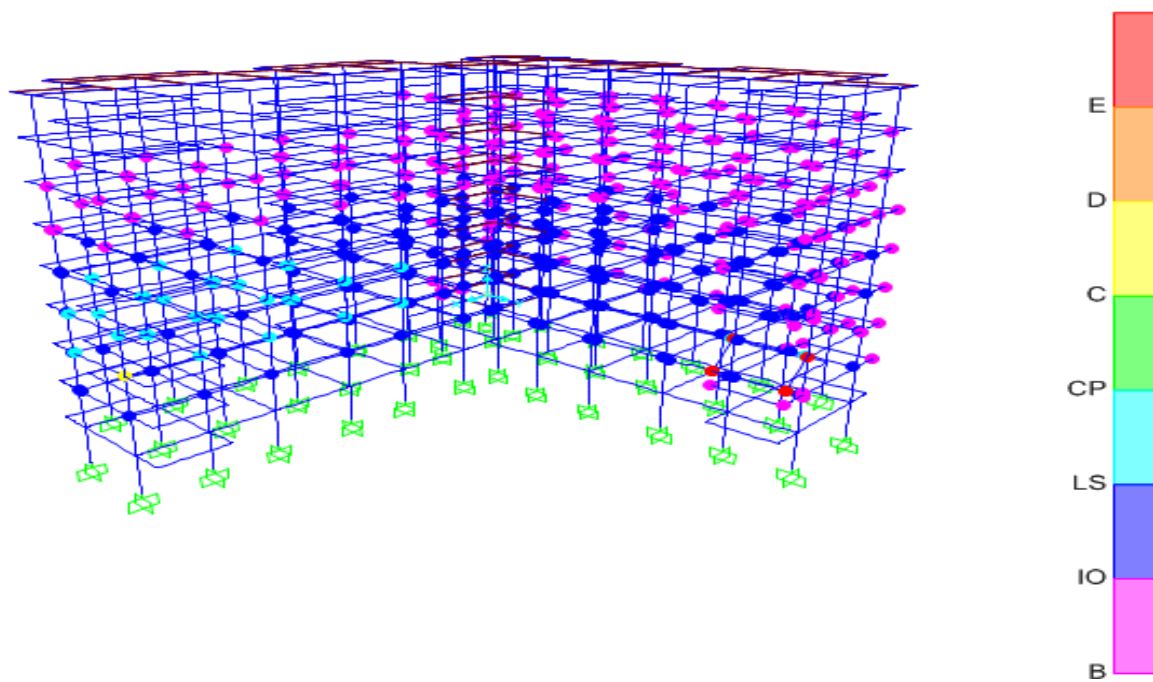


Figure 5.38. Plastic hinge distribution in eight storey L-shaped or nonlinear RC building without shear wall in Y-direction (0.05g)

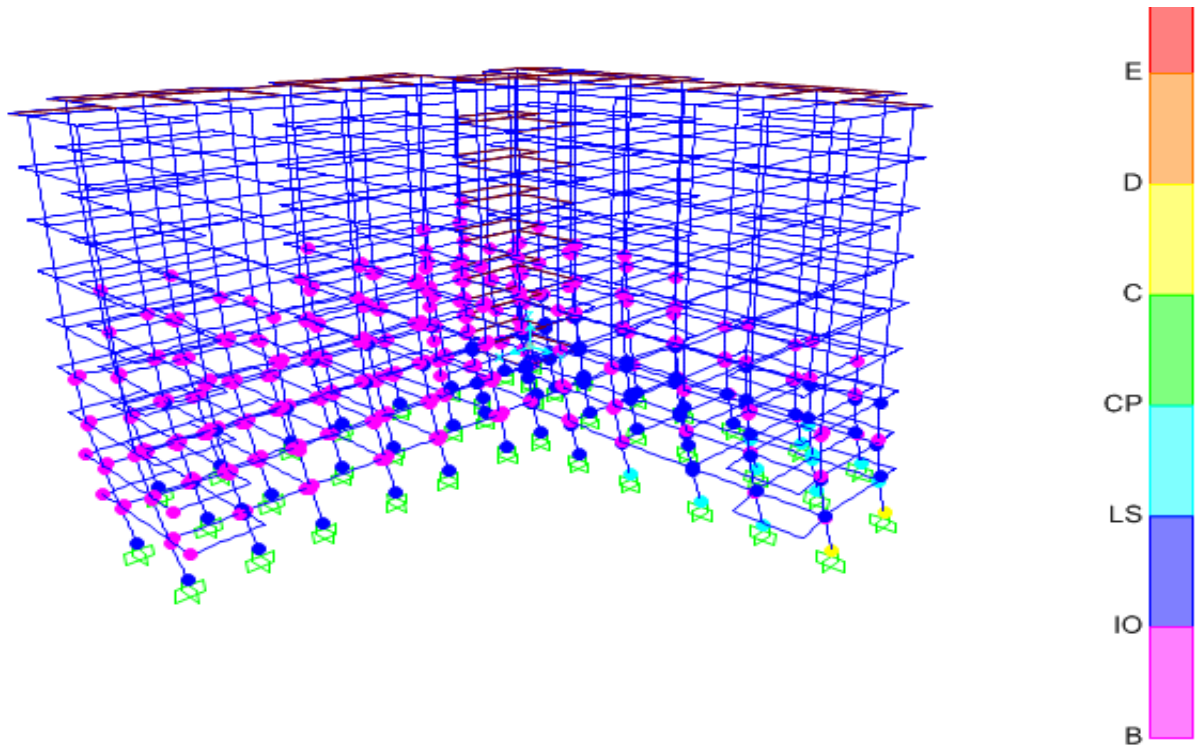


Figure 5.39. Plastic hinge distribution in eight storey L-shaped or nonlinear RC building without shear wall in X-direction (0.1g)

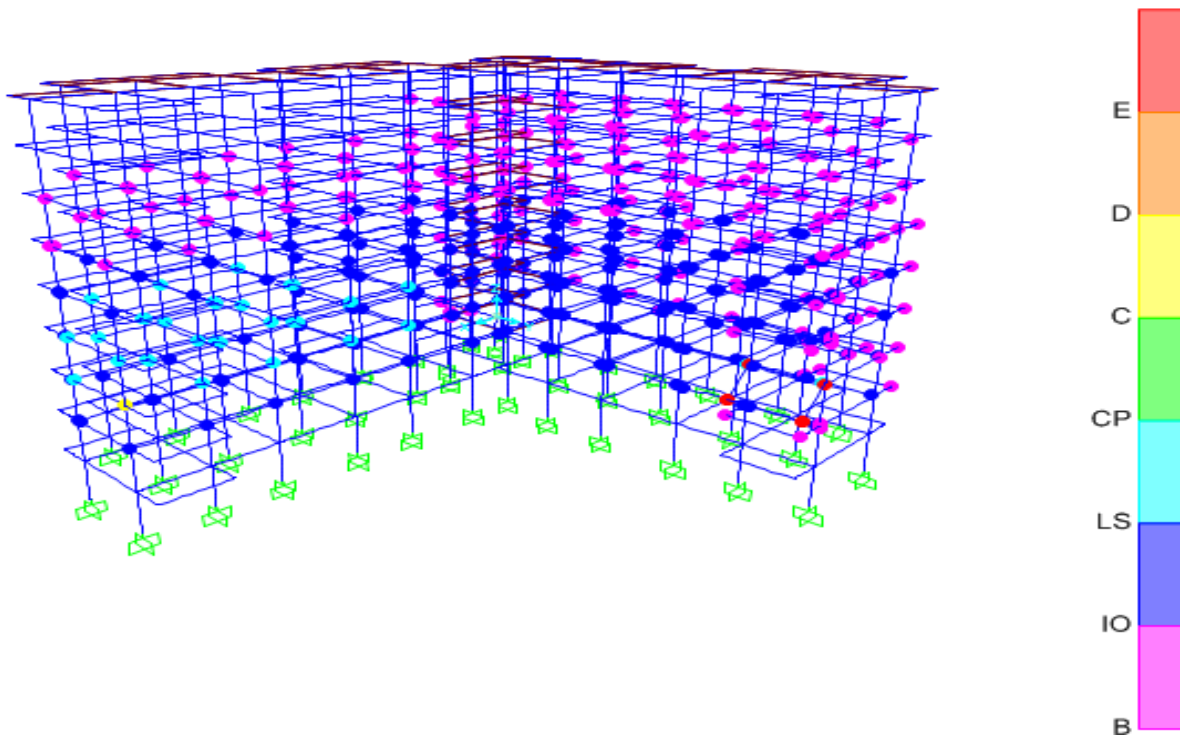


Figure 5.40. Plastic hinge distribution in eight storey L-shaped or nonlinear RC building without shear wall in Y-direction (0.1g)

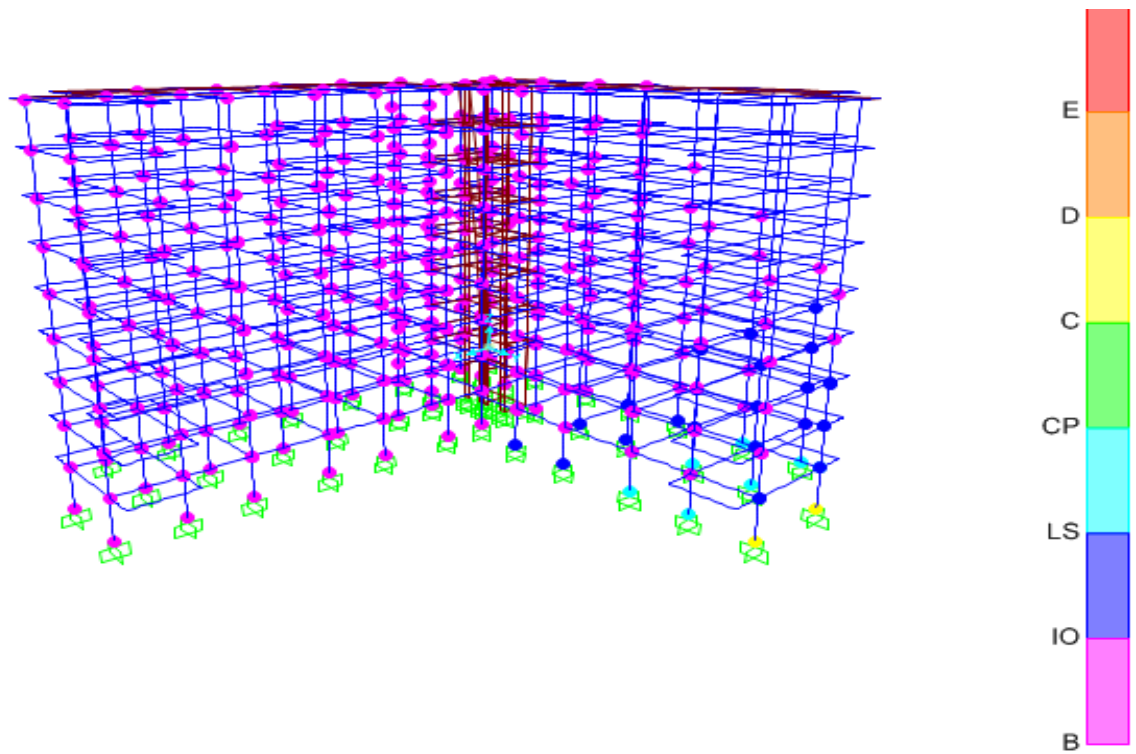


Figure 5.41. Plastic hinge distribution in eight storey L-shaped or nonlinear RC building with shear wall in X-direction(0.05g)

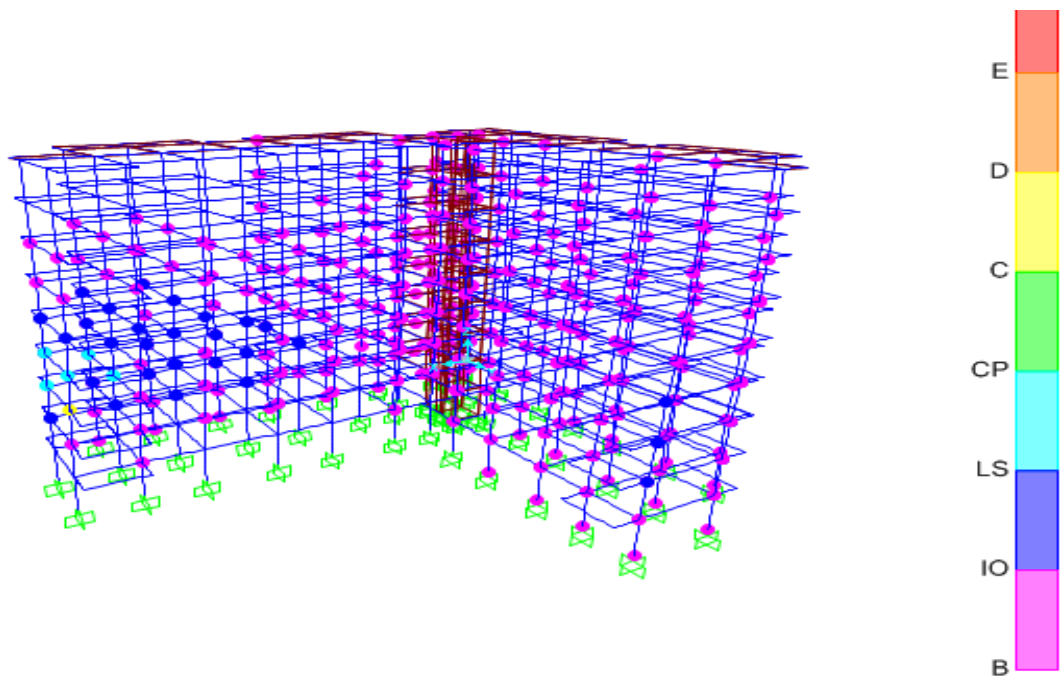


Figure 5.42. Plastic hinge distribution in eight storey L-shaped or nonlinear RC building with shear wall in Y-direction (0.05g)

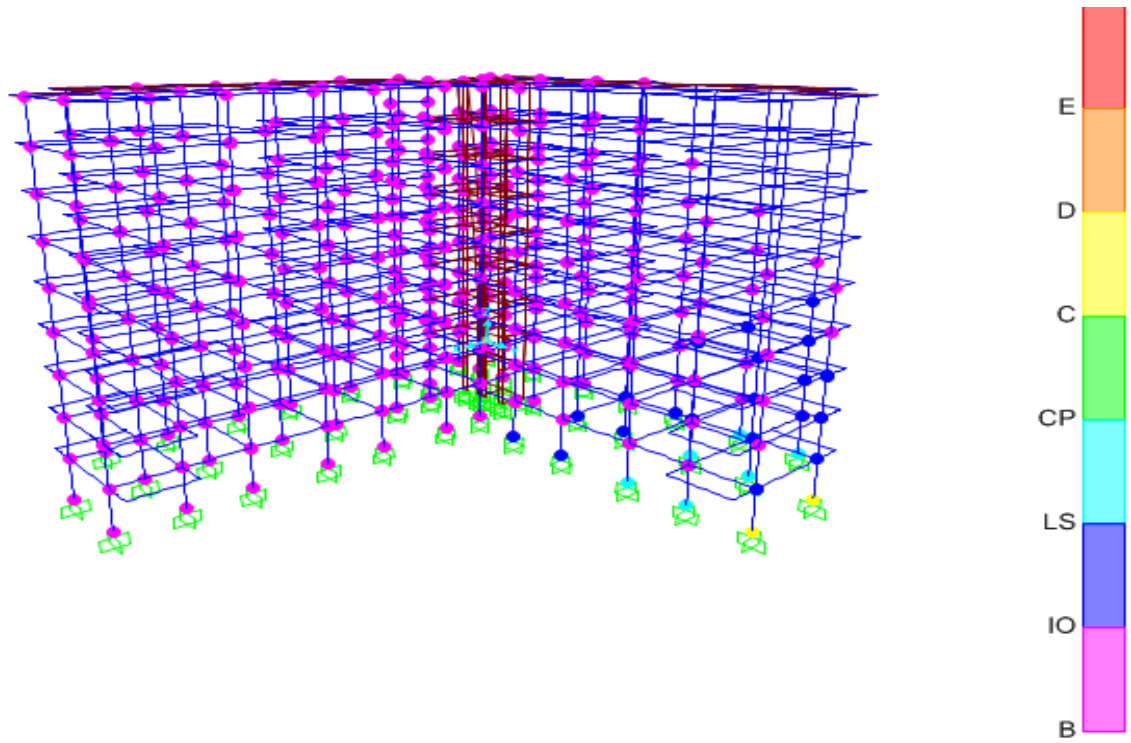


Figure 5.43. Plastic hinge distribution in eight storey L-shaped or nonlinear RC building with shear wall in X-direction (0.1g)

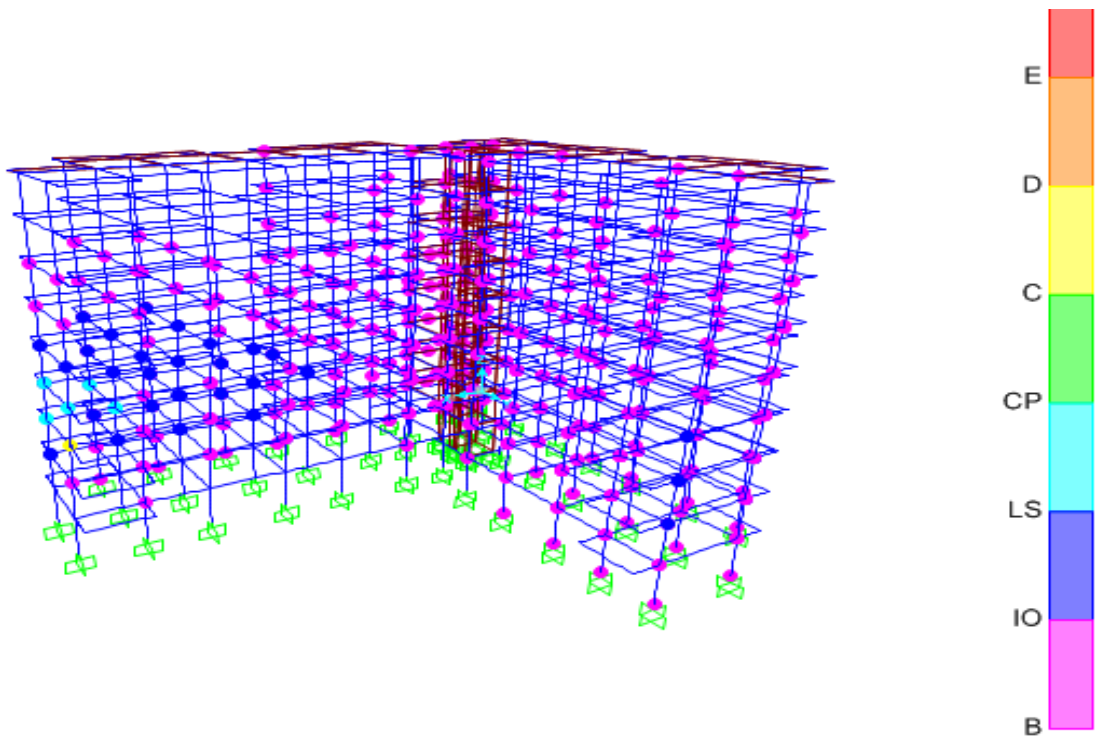


Figure 5.44. Plastic hinge distribution in eight storey L-shaped or nonlinear RC building with shear wall in Y-direction (0.1g)

### 5.11 Number of Plastic hinges at different performance level

The number of plastic hinges and their formation at target displacement for the X and Y direction, entered in to different performance level at the last step of the pushover analysis output is given in table (5.14-5.29)

Table 5.14 Number of plastic hinge formation at different performance level for eight storey  
Linear building without shear wall for push-X (0.05g)

Step	Displacement	Base Force	A-IO	IO-LS	LS-CP	>CP	Total
	mm	kN					
0	-0.111	0	2110	0	0	0	2110
1	3.718	240.0358	2110	0	0	0	2110
2	26.869	1074.8024	2110	0	0	0	2110
3	62.909	1573.2555	2088	22	0	0	2110
4	175.409	1971.6628	1731	378	1	0	2110
5	199.067	2025.3031	1717	381	12	0	2110

Table 5.15 Number of plastic hinge formation at different performance level for eight storey  
Linear building with shear wall push-X (0.05g)

Step	Displacement	Base Force	A-IO	IO-LS	LS-CP	>CP	Total
	mm	kN					
0	0.149	0	1908	0	0	0	1908
1	5.977	540.2931	1908	0	0	0	1908
2	61.782	2875.3176	1908	0	0	0	1908
3	227.476	6324.4792	1398	510	0	0	1908

Table 5.16 Number of plastic hinge formation at different performance level for eight storeys  
Linear building without shear wall push-Y (0.05g)

Step	Displacement	Base Force	A-IO	IO-LS	LS-CP	>CP	Total
	mm	kN					
0	-5.856	0	1874	0	0	0	1874
1	28.017	1746.2899	1874	0	0	0	1874
2	39.471	2212.1732	1874	0	0	0	1874
3	64.647	2619.9883	1874	0	0	0	1874
4	160.587	3329.4405	1713	161	0	0	1874
5	160.607	3276.8488	1712	161	0	1	1874
6	165.448	3319.4712	1709	164	0	1	1874

Table 5.17 Number of plastic hinge formation at different performance level for eight storeys

Linear building with shear wall push-Y (0.05g)

Step	Displacement	Base Force	A-IO	IO-LS	LS-CP	>CP	Total
	mm	kN					
0	-3.196	0	2074	0	0	0	2074
1	1.37	652.006	2074	0	0	0	2074
2	50.265	6530.9583	2074	0	0	0	2074
3	122.452	10955.955	1984	90	0	0	2074

Table 5.18 Number of plastic hinge formation at different performance level for eight storeys

Linear building without shear wall push-X (0.1g)

Step	Displacement	Base Force	A-IO	IO-LS	LS-CP	>CP	Total
	mm	kN					
0	-0.175	0	1838	0	0	0	1838
1	3.509	238.8762	1838	0	0	0	1838
2	25.98	1066.6684	1838	0	0	0	1838
3	61.382	1566.3612	1819	19	0	0	1838
4	67.632	1616.9725	1797	41	0	0	1838
5	136.382	1870.2991	1543	295	0	0	1838
6	223.882	2082.8159	1399	439	0	0	1838
7	266.171	2158.8806	1351	453	34	0	1838

Table 5.19 Number of plastic hinge formation at different performance level for eight storeys

Linear building with shear wall push-X (0.1g)

Step	Displacement	Base Force	A-IO	IO-LS	LS-CP	>CP	Total
	(mm)	(kN)					
0	0.303	0	1842	0	0	0	1842
1	5.217	437.0061	1842	0	0	0	1842
2	67.797	2922.187	1836	6	0	0	1842
3	227.471	6095.4117	1338	504	0	0	1842
4	227.491	6083.7387	1337	504	1	0	1842
5	227.615	6086.6513	1337	504	1	0	1842

Table 5.20 Number of plastic hinge formation at different performance level for eight storeys

Linear building without shear wall push-Y (0.1g)

Step	Displacement	Base Force	A-IO	IO-LS	LS-CP	>CP	Total
	mm	kN					
0	-3.196	0	2088	0	0	0	2088
1	1.371	651.7887	2088	0	0	0	2088
2	49.827	6489.9799	2088	0	0	0	2088
3	121.933	10901.4399	1998	90	0	0	2088

Table 5.21 Number of plastic hinge formation at different performance level for eight storey

Linear building with shear wall push-Y (0.1g)

Step	Displacement	Base Force	A-IO	IO-LS	LS-CP	>CP	Total
	(mm)	(kN)					
0	-3.331	0	1842	0	0	0	1842
1	3.872	1019.8914	1842	0	0	0	1842
2	26.347	3736.6685	1842	0	0	0	1842
3	57.923	5816.5771	1797	45	0	0	1842
4	57.946	5718.4641	1797	44	0	1	1842
5	63.202	6041.7117	1792	49	0	1	1842
6	63.235	5788.3248	1791	46	2	3	1842

Table 5.22 Number of plastic hinge formation at different performance level for eight storey

L-shaped or nonlinear building without shear wall push-X (0.05g)

Step	Displacement	Base Force	A-IO	IO-LS	LS-CP	>CP	Total
	mm	kN					
0	-2.402	0	2508	0	0	0	2508
1	5.381	805.8289	2508	0	0	0	2508
2	24.985	2445.8809	2508	0	0	0	2508
3	52.869	3400.407	2507	1	0	0	2508
4	79.041	3794.8477	2431	77	0	0	2508
5	124.353	4125.1283	2336	169	3	0	2508
6	139.989	4194.017	2292	209	7	0	2508
7	139.989	4194.0171	2292	209	7	0	2508
8	140.001	4194.0748	2292	209	7	0	2508

Table 5.23 Number of plastic hinge formation at different performance level for eight storey

L-shaped or nonlinear building with shear wall push-X (0.05g)

Step	Displacement	Base Force	A-IO	IO-LS	LS-CP	>CP	Total
	mm	kN					
0	-2.071	0	2522	0	0	0	2522
1	15.951	3127.8661	2522	0	0	0	2522
2	49.545	8172.9355	2522	0	0	0	2522
3	102.344	11443.5772	2519	3	0	0	2522
4	215.052	15348.4147	2021	466	35	0	2522

Table 5.24 Number of plastic hinge formation at different performance level for eight storey

L-shaped or nonlinear building without shear wall push-Y (0.05g)

Step	Displacement	Base Force	A-IO	IO-LS	LS-CP	>CP	Total
	mm	kN					
0	-0.935	0	2506	0	0	0	2506
1	34.69	3561.2234	2506	0	0	0	2506
2	77.055	7472.7801	2506	0	0	0	2506
3	233.263	12947.8002	2237	269	0	0	2506

Table 5.25 Number of plastic hinge formation at different performance level for eight storey

L-shaped or nonlinear building with shear wall push-Y (0.05g)

Step	Displacement	Base Force	A-IO	IO-LS	LS-CP	>CP	Total
	mm	kN					
0	-0.336	0	2522	0	0	0	2522
1	18.512	4677.5115	2522	0	0	0	2522
2	78.634	15683.4913	2409	113	0	0	2522

Table 5.26 Number of plastic hinge formation at different performance level for eight storey

L-shaped or nonlinear building without shear wall push-X (0.1g)

Step	Displacement mm	Base Force kN	A-IO	IO-LS	LS-CP	>CP	Total
0	-2.402	0	2508	0	0	0	2508
1	5.381	805.8289	2508	0	0	0	2508
2	24.985	2445.924	2508	0	0	0	2508
3	52.864	3400.6841	2507	1	0	0	2508
4	79.036	3796.5442	2427	81	0	0	2508
5	125.13	4134.6037	2318	184	6	0	2508
6	131.6	4166.1417	2300	201	7	0	2508

Table 5.27 Number of plastic hinge formation at different performance level for eight storey

L-shaped or nonlinear building with shear wall push-X (0.1g)

Step	Displacement mm	Base Force kN	A-IO	IO-LS	LS-CP	>CP	Total
0	-2.071	0	2522	0	0	0	2522
1	15.951	3127.8663	2522	0	0	0	2522
2	49.545	8173.2473	2522	0	0	0	2522
3	94.144	11095.4281	2521	1	0	0	2522
4	195.719	14734.6343	2054	449	19	0	2522

Table 5.28 Number of plastic hinge formation at different performance level for eight storey

L-shaped or nonlinear building without shear wall push-Y (0.1g)

Step	Displacement mm	Base Force kN	A-IO	IO-LS	LS-CP	>CP	Total
0	-0.935	0	2514	0	0	0	2514
1	34.69	3561.2291	2514	0	0	0	2514
2	77.056	7473.0099	2514	0	0	0	2514
3	231.501	12782.0847	2246	268	0	0	2514

Table 5.29 Number of plastic hinge formation at different performance level for eight storey

L-shaped or nonlinear building with shear wall push-Y (0.1g)

Step	Displacement mm	Base Force kN	A-IO	IO-LS	LS-CP	>CP	Total
0	-0.482	0	2520	0	0	0	2520
1	17.824	4604.2458	2520	0	0	0	2520
2	79.194	16024.7955	2384	134	2	0	2520

## CHAPTER- SIX CONCLUSIONS AND RECOMMENDATIONS

### 6.1 Conclusions

In the previous section base shear, storey drift, displacement, performance point, pushover curve and hinge formation for both linear and non linear structure has been compared. The result has been displayed on the graphs, figures and tables. The output shown can led to the conclusion of the following points.

- The presence of shear wall with in the building decrease the displacement as well as the base shear, generally the displacement of the building with shear wall is reduced by 56.66% relative to the displacement without shear wall for regular building. And the displacement of building with shear wall is decrease by 70.83% relative to the displacement without shear wall within the building for irregular case for both peak ground acceleration value (0.05g and 0.1g).
- The base shear of regular building with the presence of shear wall is reduced by 30.77% for both peak ground acceleration value cases. However for irregular building with the presence of shear wall is decreased by 9.09% for both peak ground acceleration value( 0.05g and 0.1g) case.
- The performance point with the presence of shear wall for regular building is differ by (27.40%, 72.55%) for displacement of structure and (50.42%, 4.8%) for base shear of structure respectively. Which work for both peak ground acceleration values (0.05g and 0.1g).
- Performance level has been reduced for irregular building relative to the regular one with and without shear wall for the two peak ground acceleration values.
- Generally increasing the value of peak ground acceleration on the new Ethiopian building code case has a negative impact on the structural performance of existing for buildings building structure, therefore they should be investigated carefully and appropriate mitigation major must be taken in order to avoid catastrophic failure during the occurrence of the expected earthquake.
- Pushover analysis is advantageous not only in predicting progressive hinge formation but also in predicting the response of the structures. Generally the progressive hinge formation can be used as a valuable input for strengthening purpose.

## **6.2 Recommendation for future Study**

- Future investigation should be done for high rise buildings.
- Shear wall arrangement and shape have significant impact on displacement and base of structure, to now further study has to be done.
- For existing buildings pushover analysis should be done for new Ethiopian building code standard and retrofitting should be the appropriate solution to reduce the impact of peak ground acceleration.
- For newly constructed structures ductility and confinement should be under the investigation of structures.
- The government should conduct more investigation and impose a performance requirement based its current priority without compromising life safety of the occupants on the existing building structures.

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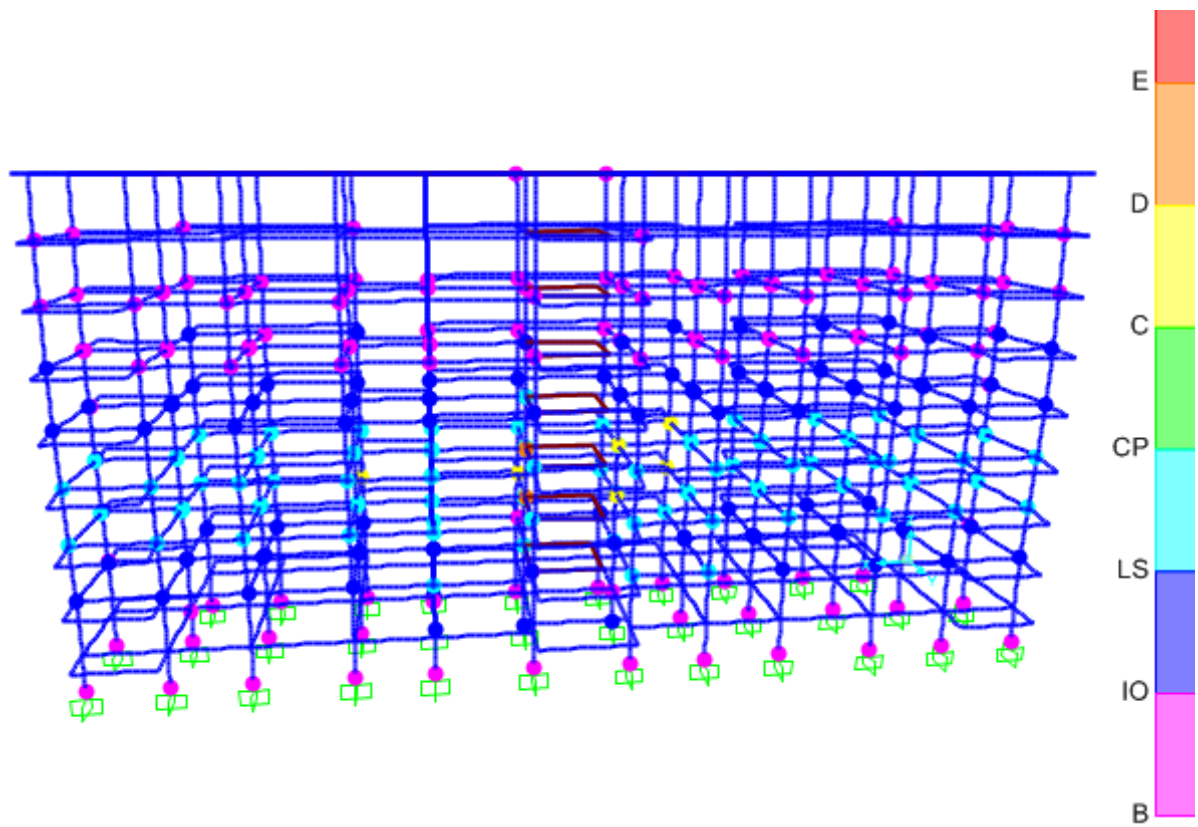
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# APPENDICES

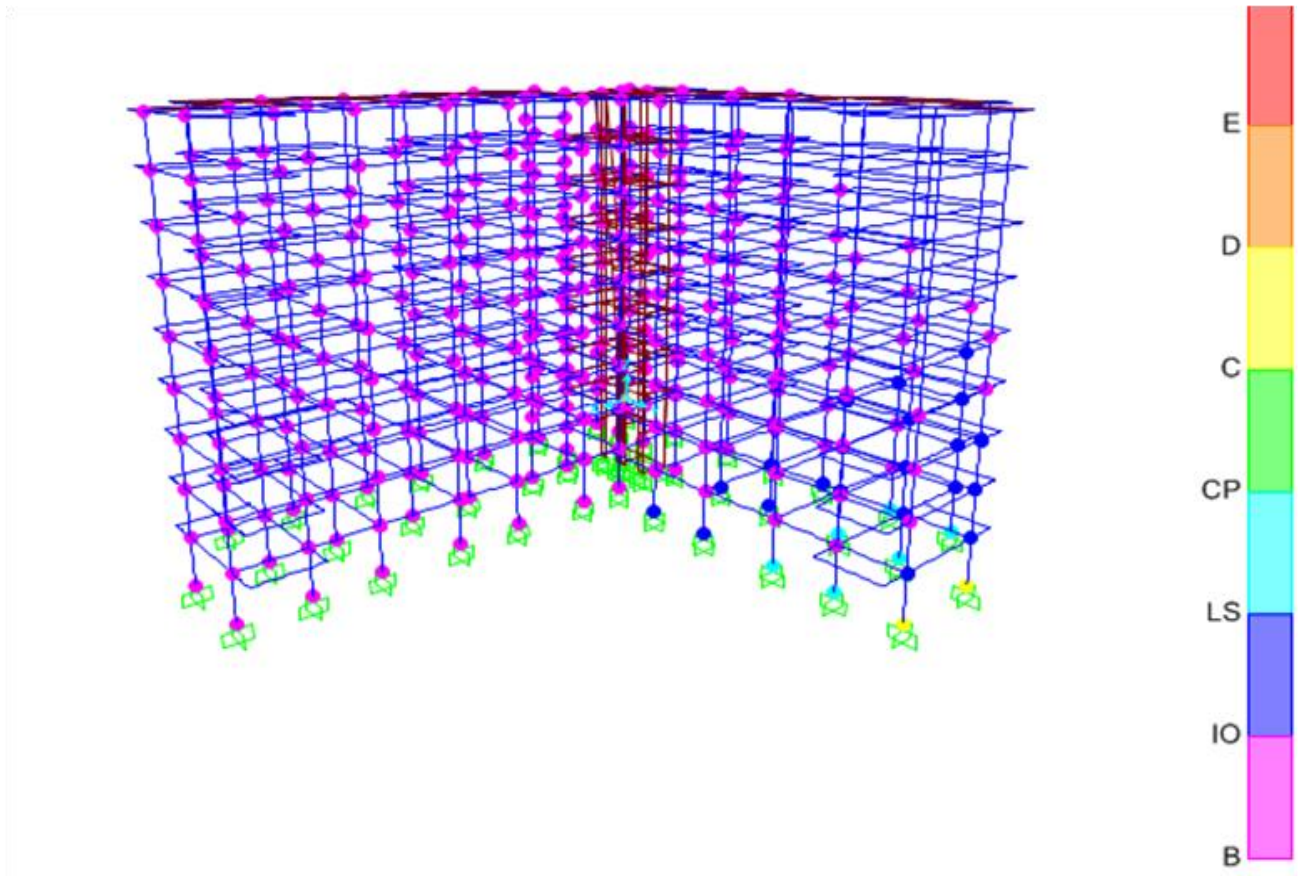
## Appendix 1

Push 9



## Appendix 2

Push 10



## Appendix 3

### Results of SAP2000 and Incremental Analysis

From the verification analysis shown figure 5.46, it is seen that in both cases the sequence of hinge formation is similar while the result of the hand calculation is similar than that of the SAP2000. In case of the incremental approach the analysis is terminated when mechanism is formed and so cannot continue to the full displacement. In case of SAP2000 analysis is terminated when mechanism is formed or when target displacement is reached.

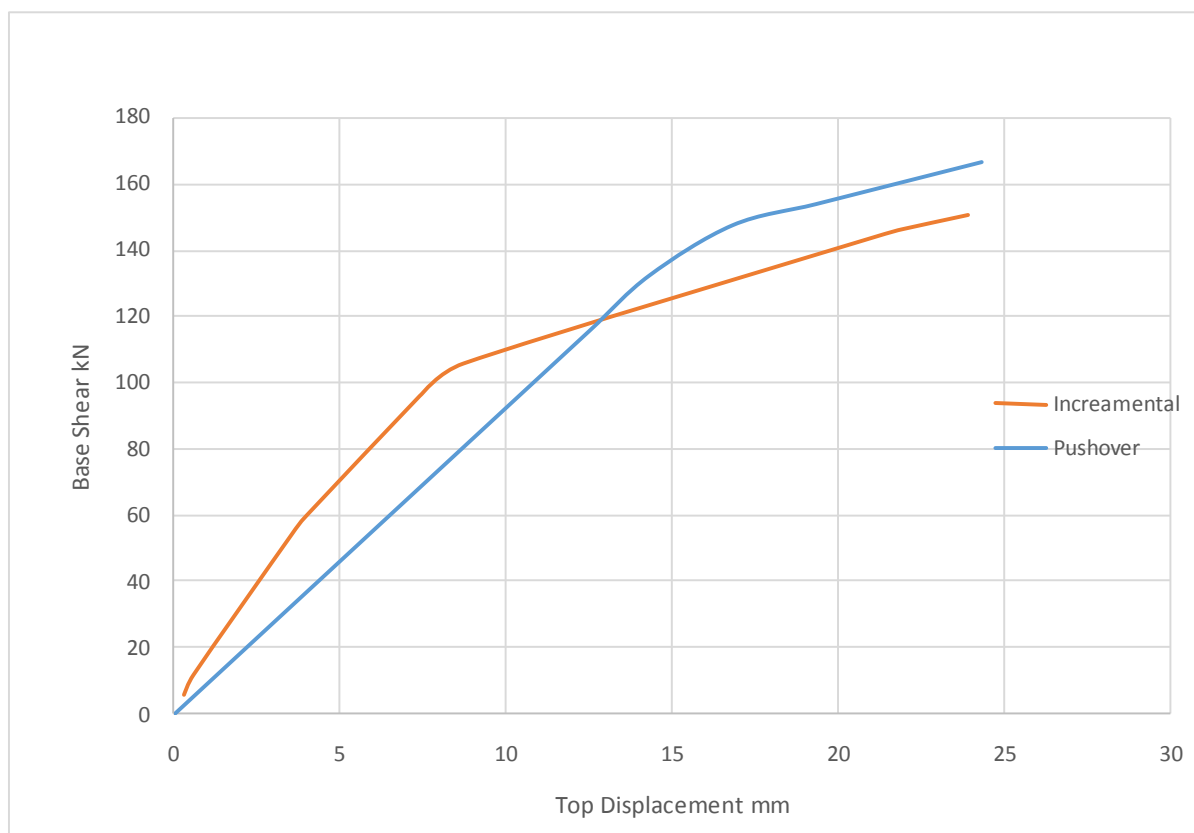


Figure 5.46 Pushover Curve For Incremental and SAP2000