STRATEGIES FOR INCLUSION OF UNCERTAINTIES IN MODELING TECHNIQUES FOR ENHANCEMENT OF CAPABILITIES OF PUSHOVER ANALYSIS

Thesis

Submitted in partial fulfillment of the requirements for the degree of

DOCTOR OF PHILOSOPHY

By

SUPRIYA R KULKARNI

(CV14F17)

Under the guidance of

Prof. K S BABU NARAYAN

DEPARTMENT OF CIVIL ENGINEERING

NATIONAL INSTITUTE OF TECHNOLOGY KARNATAKA,

SURATHKAL, MANGALORE - 575 025

JANUARY, 2019

D E C L A R A T I O N

By the Ph.D. Scholar

I hereby *declare* that the Research thesis entitled "**Strategies for Inclusion of Uncertainties in Modeling Techniques for Enhancement of Capabilities of Pushover Analysis**" which is being submitted to the **National Institute of Technology Karnataka, Surathkal** in partial fulfillment of the requirements for the award of the Degree of **Doctor of Philosophy** in **Civil Engineering** is a *bonafide report of the research work carried out by me*. The material contained in this Research thesis has not been submitted to any University or Institution for the award of any degree.

Place: NITK, SURATHKAL

Date:

SUPRIYA R KULKARNI

148014 CV14F17

(Research Scholar)
Department of Civil Engineering

C E R T I F I C A T E

This is to certify that the Research thesis entitled "**Strategies for Inclusion of Uncertainties in Modeling Techniques for Enhancement of Capabilities of Pushover Analysis**" submitted by **Supriya R Kulkarni** (Register Number: **148014CV14F17**) as the record of research work carried out by her, *is accepted as Research thesis submission* in partial fulfilment of the requirements for the award of degree of Doctor of Philosophy.

Prof. K. S. BABU NARAYAN
Research Supervisor
Department of Civil Engineering

Chairman – DRPC
(Signature with Date and seal)

DEDICATED

TO

MY PARENTS,

FAMILY MEMBERS, TEACHERS

AND

FRIENDS

ACKNOWLEDGEMENT

First and above all I would like to thank the almighty for bestowing upon me the choicest blessings and providing me the capability to proceed successfully in my life.

Since its inception, NITK Surathkal has always been a harbinger of educational excellence and research throughout the country. I am extremely fortunate to be associated with this great institution.

I take immense pleasure to express my sincere gratitude to my research supervisor Prof. K. S. Babu Narayan for his motivation and invaluable guidance throughout my research work. His profound subject knowledge and skillful ways of problem solving are truly for everyone’s admiration. I am grateful to him for his keen interest in the preparation of this thesis. It has been my pleasure to work with him.

I take immense pleasure to express my sincere gratitude to my research supervisor Prof. K. S. Babu Narayan for his motivation and invaluable guidance throughout my research work. I am grateful to him for his keen interest in the preparation of this thesis. It has been my pleasure to work with him.

I acknowledge my sincere thanks to Prof. N. Lakshman, Dept. of Applied Mechanics and Dr. A.S. Balu, Dept. of Civil Engineering for being members of Research Progress Assessment Committee and giving valuable suggestions and encouragement at various stages of this work.

I wish to thank Prof. Katta Venkataramana, former Dean NITK Surathkal, Prof.K.N Lokesh, former Head of the Civil Engineering Department, for their support and encouragement throughout my stay at the NITK campus.

Head of Civil Engineering Department Prof. Varghese George, Dr. B.Manu Seceratry DRPC, Prof. Subhash Yaragal and Dr. Sunil B M, have continuously encouraged, helped and supported me for which I am thankful to them.

I also extend my gratitude to all the teaching faculty and supporting staff of the Civil Engineering Department, NITK for their support and cooperation provided during the research work. I am also thankful to all my friends Archana, Chaithra, Prachi, Rashmi, Priyanka, Manoj for their continuous encouragement and help during my stay at NITK.

I am especially grateful to my father Sri. Ravindra V Kulkarni and mother Smt. Rajani, who provided me the best available education and encouraged in all my endeavors. I am grateful to my brother Mr. Atul Kulkarni, sister in law Mrs. Smitha Kulkarni and my best friend Mr. Dnyneshwar Chougule, for their cooperation and moral support, and I lovingly acknowledge the support and help extended from my family members during this research work. They have always been a source of inspiration for me.

Lastly I am grateful to everybody who helped and encouraged me during this research work.

Thank you one and all, it was indeed an honor being here.

NITK Surathkal **SUPRIYA R KULKARNI**

Date:

ABSTRACT

Pushover analysis is an extensively used tool for performance evaluation of structures under seismic loads. Continuous efforts are on for enhancement of capabilities of the same. Refinements to geometric and material modeling have led to better understanding of structural performance. Notwithstanding the attempts, though analytical predictions of strengths have been in close agreement with experimental, displacements predicted have differences. Attempts to close this gap between predicted and observed displacement characteristics have always been centered around geometric and material modeling. The sequence of plastic hinge formation and its influence on displacement characteristics needs very serious consideration. The present study highlights this issue with illustrations by suggesting strategies to reduce tremendous computational efforts, required for considering plastic hinge formation sequence in performance appraisal.

Strategy 1 considers 15% of potential plastic hinge locations and allows variations leading to different sequences. Two sub strategies have been proposed by way of allocation of defective plastic hinge locations to horizontal and vertical planes. When hinges with uncertainties are restricted to horizontal planes, variations in base shear values are between 1.4% to about 1.7% in comparison with experimental study. Whereas displacements are lower by 10%. When hinges with uncertainties are restricted to vertical planes, the difference in both base shear and displacements show the range of variations of 7% and 15% with respect to experimental results suggesting assignment of plastic hinges with uncertainties distributed in horizontal planes for better results. Strategy 2 adopts randomization of plastic hinge locations with uncertainties associated, distributed throughout the 3D frame. The analysis results indicate such a consideration is superior then strategy 1 to get analytical results almost perfectly matching with experimental results.

Strategy 1, finds application where defect and deficiency features are known a priori. Whereas strategy 2 can be employed in all situations.

Keywords: Pushover analysis; Uncertainties; Sequence of plastic hinge formations; Performance appraisal.

CONTENTS

LIST OF FIGURES

LIST OF TABLES

NOTATIONS

V	:	Force
δ	:	Displacement
S_a	:	Spectral acceleration
S_d	:	Spectral displacement
Z_X	:	Sectional modulus
Z_P	:	Plastic section modulus
E	:	Elastic modulus
M_Y	:	Yield moment
M_P	:	Plastic moment
W	:	Point load
W_c	:	Collapse load
l	:	Span
M	:	Moment
W_A	:	Additional load
f_s	:	Steel strength
f_c	:	Concrete strength
d_c	:	Effective cover
h	:	Height

CHAPTER 1

INTRODUCTION

1.1 PHILOSOPHY OF EARTHQUAKE RESISTANT DESIGN

Mitigation of damage to structures and minimization of loss of life and property due to earthquakes is the main objective of earthquake engineering. Severity of ground shaking at a given location during an earthquake can be minor, moderate or major. Minor quakes are more frequent and structures are designed and detailed to perform elastically with no damage to the structure as a whole and the component parts. Slight damage to structures and components that can be repaired after a moderate quake allows elastoplastic behavior. Design and detailing must allow for accommodation of large deformations of structures and components without collapse due to major quakes. This essentially is possible if plastic behavior of materials and elements is appropriately understood. The characterization of the various performance levels, hence, has led to performance-based earthquake engineering.

1.2 PERFORMANCE BASED DESIGN

Performance based design approach is being adopted by seismic design codes and is perhaps the universally accepted process of design against seismic excitations. A flow chart that presents the key steps in the performance-based design process is shown in Fig 1.1. It is an iterative process that originates with the selection of performance objectives, followed by the development of a preliminary design. An assessment to check the design for compliance with performance objectives is done to ascertain requirement of redesign and reassessment. All these steps are repeated till desired performance levels are accomplished satisfying iteration termination criteria. Investigation of performance of structure requires adoption of inelastic analytical procedures that help to know the actual behavior of structures by recognizing failure modes and the potential for progressive collapse. These procedures mostly include inelastic time history analysis and inelastic static analysis. Inelastic static analysis is also known as pushover analysis. The need for pushover analysis also arises from the fact that the existing buildings can become seismically deficient as seismic design code necessities are constantly elevated with advancement in engineering knowledge. Though the inelastic time history analysis is considered suitable to predict the force and deformation demands the use of this method is limited because dynamic response is very sensitive to modelling and ground motion characteristics. Due to the prohibitive computational time and effort required to perform a complete nonlinear dynamic analysis, researchers are showing keen interest in nonlinear static pushover analysis. In recent decades, pushover analysis has been verified to be a strong tool for performance assessment of buildings at different design levels.

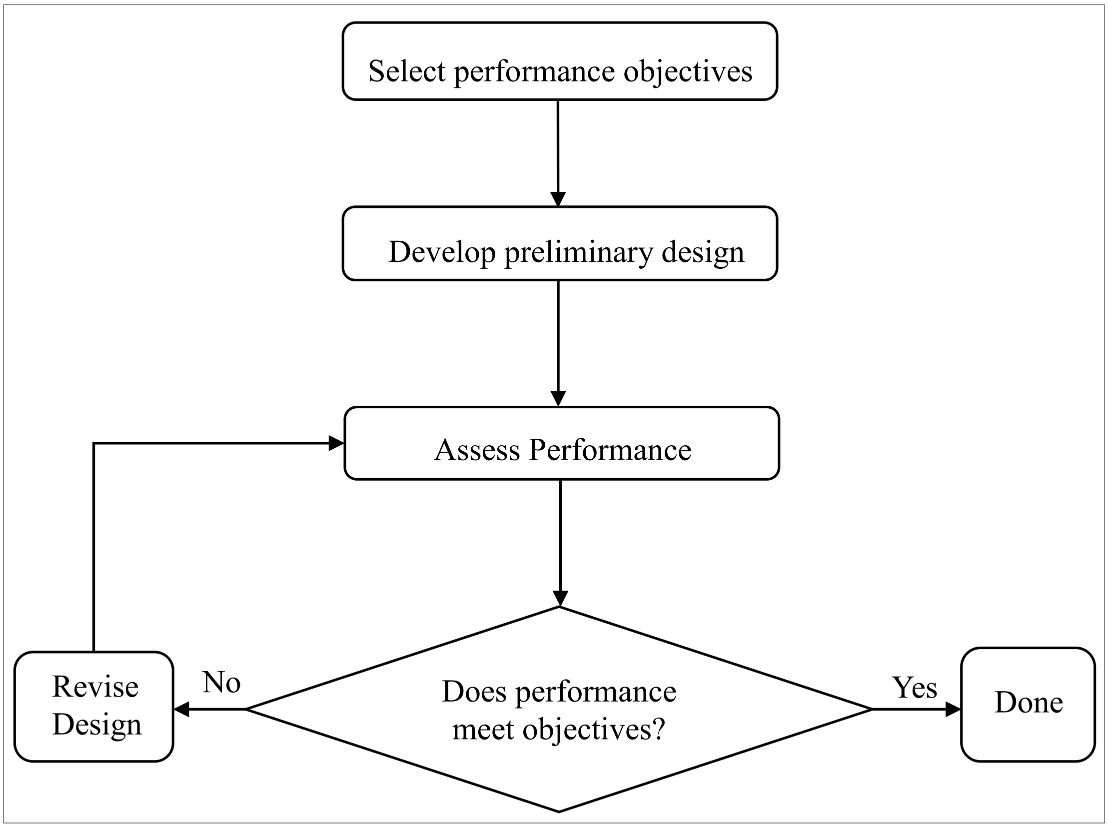


Fig. 1.1 Performance-Based Design Flow Diagram (ATC, 1997a)

1.3 PUSHOVER ANALYSIS

Pushover analysis is a nonlinear static analysis used to determine the force – displacement relationship, or the capacity of the structure. Pushover curve represents the lateral capacity of the building by plotting nonlinear relation between base shear and displacements. Fig 1.2 shows the schematic representation of pushover analysis concept. Applied Technological Council document (ATC 40) and Federal Emergency Management Agency document (FEMA 273 and FEMA 356) give guidelines for performing pushover analysis.

In pushover analysis the structure is subjected to monotonically increasing lateral forces or displacements (force controlled or displacement controlled) with an invariant pre-fixed height-wise distribution until the peak response (limit state of collapse) is reached. Peak response can be either maximum capacity at instability or at a predefined performance level of force or displacement. Reliable post yield material models and information regarding inelastic member deformations are extremely important for obtaining meaningful results in pushover analysis.

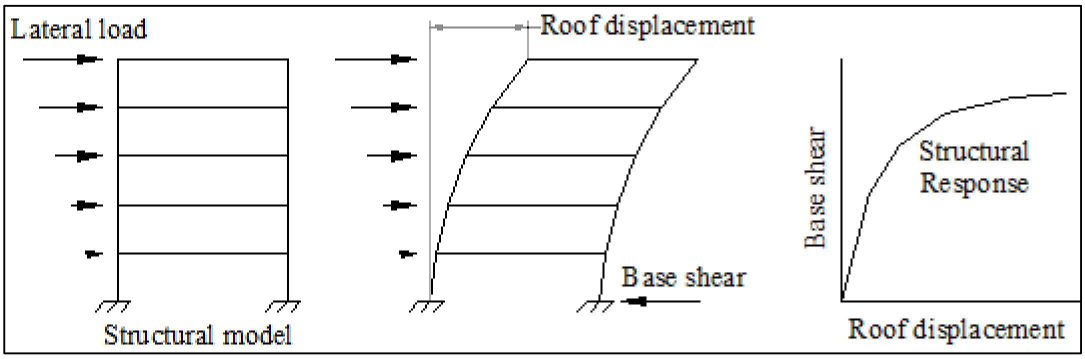


Fig. 1.2. Schematic representation of pushover analysis concept

Three primary parameters that are obtained from pushover analysis are capacity, demand and performance.

Capacity

Central to the idea of pushover analysis is the generation of the pushover curve which is also called the capacity curve. It presents the relationship between lateral force and corresponding displacement. As generation of pushover curve is independent of method used to calculate demand, availability of pushover curve provides valuable insights to structural performance. A typical capacity curve which can be obtained from pushover analysis is shown in fig 1.3.

Capacity of a structure depends on the strength and deformation characteristics of individual components of the structure. Pushover analysis employs a series of sequential elastic analyses, superimposed, to obtain force – displacement capacity diagram of overall structure. Reduced resistance of yielding components is captured to assess behavior of the structure against lateral loads.

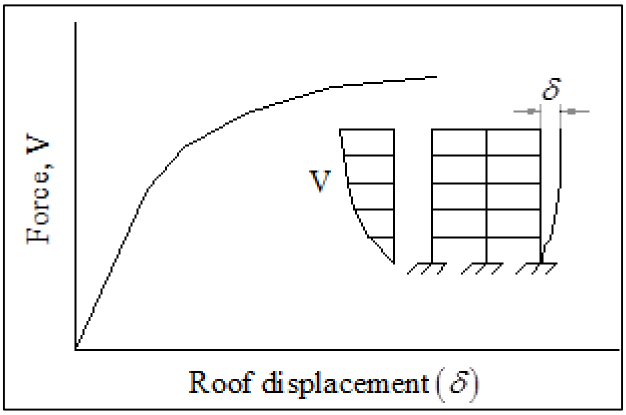


Fig. 1.3. Capacity curve

Demand

Demand is the representation of the earthquake ground motion. Ground motions during an earthquake produce complex horizontal displacement pattern in structures that vary with time. In nonlinear static analysis procedure, demand is represented by an estimation of the displacements or deformations that the structure is likely to undergo. This is in distinction to conventional linear elastic analysis method in which demand is represented by prescribed lateral forces applied to the structure. Fig 1.4 shows typical demand curve.

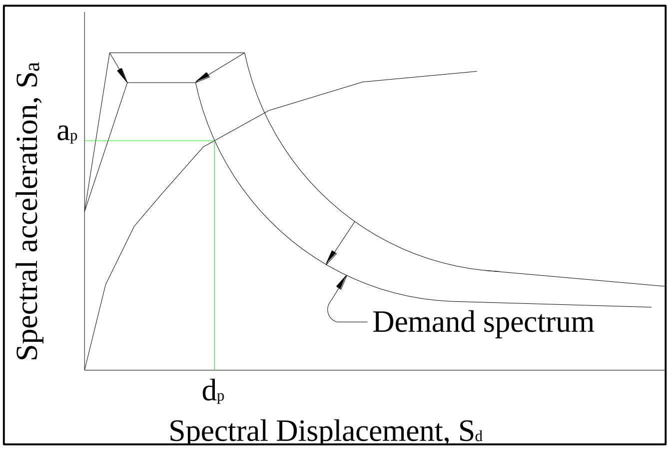


Fig. 1.4. Demand Curve

Performance

Performance check confirms that structural and non-structural components are not damaged beyond acceptable limits (Strength and Serviceability limits) of the performance objective. Performance objectives are preset levels related to damage state of building and its components.

Performance point is the point where demand and capacity curves intersect each other. Fig 1.5 represents seismic safety evaluation based on performance levels. The zone where the performance point lies indicates availability or exhaustion of structure's capacity.

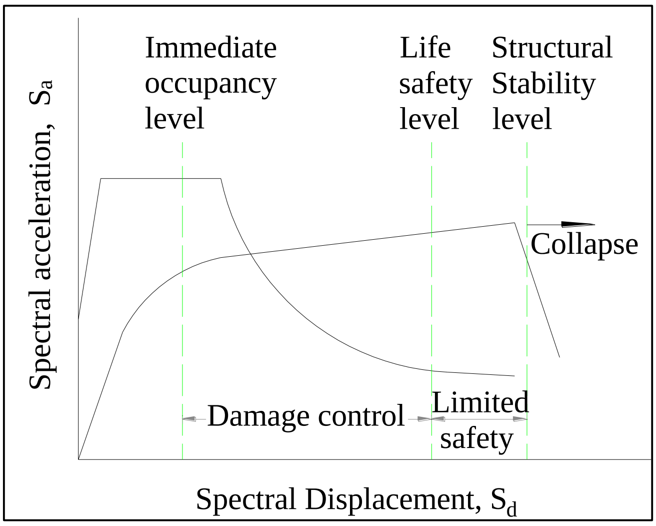


Fig. 1.5. Seismic safety evaluation based on performance level.

Three distinct structural performance levels are defined as detailed below,

Immediate occupancy level (IO)

Structural performance level immediate occupancy means the post-earthquake damage state in which merely partial structural damage has occurred. The basic vertical and lateral force resisting systems of the building preserve nearly all of their pre – earthquake strength and stiffness. The risk of life threatening injury as an outcome of structural damage is very low, and although some minor structural repairs may be appropriate, these would generally not be required prior to re-occupancy.

Life safety level (LS)

Life safety level of structural performance suggests post-earthquake damage state in which substantial damage to the structure has occurred, but some margin against either partial or total structural collapse remains. Some structural elements and components are rigorously damaged, but this does not result in the large falling debris hazards, either within or outside the building. Injuries may occur during the earthquake; however, it is likely that the overall risk of life-threatening injury as a result of structural damage is low. It should be possible to repair the structure; however, decision on repair shall depend on economic considerations.

Collapse prevention level (CP)

When structure is on the verge of partial or total collapse the structural performance level is termed as collapse prevention. Extensive damage to the structure will have occurred, potentially including significant degradation in the stiffness and strength of the lateral force resisting system, large permanent lateral deformation of the structure and to an extent degradation in vertical load carrying capacity. However, all major components of the gravity load resisting system must continue to carry their gravity load demands. Significant risk of injury due to falling of debris hazards may exist. The structure may not be subjected to repair and is unsafe for re-occupancy, as aftershock activity could induce collapse.

1.3.1 Force controlled and displacement controlled procedure

Pushover analysis can be carried out as either force-controlled or displacement-controlled depending on the physical nature of the load and the behavior anticipated from the structure. Force-controlled option is used when the load is known and the structure is expected to be able to support the load. In force-controlled pushover procedure some numerical problems that affect the accuracy of the results occur, since target displacement may be associated with a very small positive or even negative lateral stiffness because of the development of mechanisms and P-delta effects. Hence, displacement controlled pushover analysis is preferred to overcome these problems. Displacement controlled procedure is also used when specified drifts are sought (such as in seismic loading), where the magnitude of the applied load is not known a priory, or when the structure can be expected to lose strength or become unstable. The roof displacement at the center of mass of the structure is chosen as control displacement.

Specialized and customized codes are available for the analyst to perform pushover analysis that involves following steps,

1. A two or three dimensional model that represents the overall structural behavior is created.
2. Bilinear or trilinear load-deformation diagrams of all significant members that affect lateral response are defined.
3. Gravity loads composed of dead loads and specified portion of live loads are applied to the structural model firstly.
4. A predefined lateral load pattern (in load controlled analysis) or displacement profile (in displacement controlled analysis) which is distributed along the height of the building is applied.
5. Lateral loads or displacements as the case may be are increased and the behavior of the structure is tracked for base shear, roof displacement and yielding characteristics at each interval of load or displacement.
6. The structural model is modified to account for the reduced stiffness's at each load or displacement.
7. Results of each step are superimposed with previous steps at the end of each step.
8. Lateral loads or displacements are incremented and analysis is carried forward until structure becomes unstable or predetermined level of force or displacement is reached
9. Base shear is plotted against displacement to get the global capacity curve also known as pushover curve.

1.4 ADVANTAGES AND LIMITATIONS OF PUSHOVER ANALYSIS

Modelling, analysis procedure and acceptance criteria that are recognized are available for design and evaluation of structures. Pushover analysis exposes design deficiencies that may remain hidden in an elastic analysis such as story mechanisms, excessive deformations demands, strength irregularities. The analysis provides information on the sequence of yielding and also helps in visualization of mode of failure.

Though pushover analysis finds extensive usage in performance appraisal, there are questions about its capabilities in addressing prediction behavior for modes higher than the first. Difficulties exists in its utility for analysis of structures where mass and stiffness distributions are irregular. As has been pointed out earlier, analysis results are very sensitive to geometry and material models and uncertainties associated with strength and stiffness determination. Research is continuously on in terms of experimental and analytical investigations for enhancement of capabilities of pushover analysis. The technique is continuously being refined and redefined to make behavior prediction more close to reality.

1.5 ORGANIZATION OF THE THESIS

"Strategies for inclusion of uncertainties in modeling techniques for enhancement of capabilities of pushover analysis" is an analytical investigation that highlights the influence of plastic hinge formation sequence on analysis results and suggest strategies for its consideration and inclusion. The findings of the investigations have been organized as under

A brief overview of the philosophy of earthquake resistant design, performance based design, need, advantages and limitations of pushover analysis are presented in chapter 1.

Summary of literature relating to origin of pushover analysis, improvements over conventional methods of analysis, issues to be resolved are presented in chapter 2, giving details of the need, objectives and scope of present investigation.

Influence of hinge formation sequence on pushover analysis results has been illustrated with an example of a propped cantilever beam and importance of consideration of hinge formation sequence in pushover analysis has been elaborated in chapter 3.

Chapter 4 discusses the variability of pushover analysis results due to variation in strength characteristics and detailing deficiencies. Details of investigations for highlighting the overriding influence of hinge formation sequence over other uncertainties have been presented.

Difficulties associated with inclusion of plastic hinge formation sequence in pushover analysis have been explained and formulation of strategies for consideration and inclusion for enhancement of behavioral prediction capabilities have been presented in chapter 5.

Chapter 6 provides conclusions, research outcomes and contributions of present investigation.

CHAPTER 2

LITERATURE REVIEW

2.1 CHAPTER PROLOGUE

The origin, growth and development of pushover analysis has been presented here, offering an insight to the attempts made by researchers in refining, redefining and enhancing this analysis tool in performance based designs.

2.2 ORIGIN OF INELASTIC STATIC PUSHOVER ANALYSIS

Use of inelastic static analysis in earthquake engineering is traced to be the work of Takeda and Sozen(1970), where a realistic conceptual model for envisaging the dynamic response of RC member has been studied based on static force displacement relationship which reflects the changes in stiffness for loading and unloading the member. Gulkan and Sozen (1974) derived a single degree of freedom system to represent the multi degree of freedom via an equivalent or substitute structure for evaluation of load displacement curve by further analysis to obtain initial and post yield stiffness, yield strength and ultimate strength.

2.3 IMPROVEMENTS OVER CONVENTIONAL PUSHOVER METHODS

Improvements to conventional methods were attempted by Shibata and Sozen(1976) by proposing modified linear model for estimation of effect of energy dissipation in the nonlinear range. This resulted in availability of a procedure at the level of linear spectral response analysis with explicit options for levels of inelastic response. Simplifications to inelastic analysis procedure for MDOF system were proposed by Saiidi and Sozen (1981), Fajfar and Fischinger (1988).

Deierlein and Hsieh (1990) have exploited the Capacity Spectrum method to compare the experimental and theoretical results for the seismic response of a single storey single bay steel frame with the analytical results of 2D pushover analysis. The frame has been modeled with semi-rigid connections and the results obtained have shown differences ranging from 10% to 20% for period of vibration, maximum displacement and maximum acceleration. It has been recognized that the Capacity Spectrum method could provide reasonably accurate lower and upper bounds on the inelastic response of a structure subjected to strong ground motion. Utilization of the Capacity Spectrum Method in four case studies of structures to evaluate their seismic response has been carried out by Mahaney et al. (1993). The structures analyzed include one-storey and two-storey wood-frame residences, an eleven-storey reinforced concrete shear wall building and several framed buildings with brick infilled walls. ADRS format has been adopted. Results have showed that the damped elastic earthquake displacement demands need not necessarily be equal to the actual inelastic displacement demands assumed. This mismatch has been attributed to the short predominant periods of some of the structures which do not obey the equal displacement rule. However the damage predicted has agreed with the observed damage for the eleven-storey reinforced concrete shear wall building in this study.

General assessment of pushover analysis on 2-, 5-, 10-, and 15- storey steel moment resisting frames has been carried out by Lawson, Vance and Krawinkler (1994). The pushover analysis results have been compared with those obtained from nonlinear dynamic analyses for seven ground motions. Storey deflections calculated from the pushover analyses have correlated well with those resulting from nonlinear dynamic analyses for the short structures. Additionally weak stories that have led to concentration of inelastic deformations have been identified. Results for deflection of structures from nonlinear static and nonlinear dynamic analysis differ owing to sensitivity of results to applied load patterns. However for short structures good correlation has been obtained for inter-storey drifts from the pushover and nonlinear dynamic analyses. It has been observed that accuracy of inter storey ductility ratios and plastic hinge rotations evaluation is poor for tall structures especially at higher levels. It has also been reported that area under static load displacement curve was not a good measure of cumulative damage demand as the correlation with dynamic hysteretic energy dissipation was poor.

Kilar and Fajfar (1997) have developed a pseudo three dimensional model of the structure that helps in behavioral prediction in the inelastic range. Assemblages of two-dimensional macroelements/substructures such as frames, walls and coupled walls have been adopted to model the structures.

N2 method, a combination of pushover analysis of MDOF model with response spectrum of an equivalent SDOF system has been developed by Fajfar (2000). A.S.Moghdam et al, (2000) have proposed a response spectrum based pushover procedure to obtain seismic response estimates of asymmetrical buildings systems. The procedure also helps in inclusion of the 3-D effects caused by the response torsion.

A comparative study on inelastic static pushover analysis and inelastic dynamic analysis on 12 RC building frames with different characteristics has been conducted by Elnashai (2001). Natural and artificial earthquake records have been used for the analysis. It has been observed that static pushover analysis is more appropriate for low rise and short period framed structures and for buildings with structural irregularities, static pushover analysis has yielded good correlation with dynamic analysis. One of the first examples of modal pushover analysis procedure for estimating the seismic demand of buildings is the study by Chopra and Goel (2001). Modal pushover analysis on a 9 storey steel frame building, to determine peak inelastic response for comparison with nonlinear response history analysis has shown that analysis is more accurate for practical application in building design and evaluation.

Transformation of MDOF system to an equivalent SDOF system for evaluation of buildings has been proposed by Bagchi(2004). Kunnath et al (2004) have evaluated seismic performance of non-ductile reinforced concrete frame buildings in low to moderate seismic force regions. The detailing deficiencies have been introduced and investigated. The study revealed that moderate earthquakes caused no severe damage and damages to beams were more than columns except in lower stories of structures.

A new multi-modal pushover method has been introduced by Barros and Almeida (2005) where in the load pattern based on the basis of relative participation of each mode of vibration is possible for inclusion. The study outcomes highlights the importance of higher mode effects and also suggest need for experimental investigations for validation of computational results.

Displacement coefficient method of FEMA-356 has been adopted by N. Lakshmanan (2006) assigning weightage factor for hinges in beams and columns for performance ranges, and vulnerability indices as performance indicators have been determined. Rofooei (2007) has employed adaptive pushover method for improvement of accuracy of analysis results and concludes that the adaptive load pattern is more successful for estimation of response parameters of the structural models.

Redistribution of inertia forces after the yielding of the structure was proposed by Jianmeng, et al. (2008) in improved modal pushover analysis. The procedure helps in contributions of higher modes and also address effect of the redistribution of inertia forces after yielding, and making estimates of responses more accurate. Performance appraisal of reinforced concrete frames using pushover analysis has been studied by P.Poluraju et al (2011). Analysis indicates that appropriate detailing of reinforced concrete frame building can force desired modes of failure.

Investigations by K.Rama Raju et al (2012) has highlighted importance of appropriate study of stress strain curves for material and moment curvature relationships for hinge characteristics. Sofyan.Y.Ahmed (2013) has investigated seismic hazards of RC frame buildings in Iraq and analysis has brought out the importance of strong column and weak beam idea in seismic resistant design. Similar studies have been done by M. Mouzzoun et al (2013) for compliance of RC structures to moroccan seismic code RPS200.

Investigations by Hirde Suchita et al (2013) on RC framed buildings with masonry infills quantify the contributions of infills to lateral stiffness, strength and their influence on overall ductility and energy dissipation capacity. Comparison of behavioral characteristics of bare frame, infilled frames and frames with weak storey has been done by Raut Nivedita N. et al (2013). Effects of variation of infill layout has also been investigated. Changes in displacement characteristics due to presence or absence of infills have been determined. Sharma Akanshu et al. (2013) have reported pushover test results on a full scale non-seismically detailed RC structure which was replica of portion of an existing structure having mass and stiffness irregularities. Large variations of test results from those predicted by analysis have been observed. Suggestions for better geometric and material modeling have been provided.

Cyclic lateral force distribution established based on the mode shapes and prescribed displacement history for performance evaluation of RC building has been attempted by Panyakapo Phaiboon (2014) by adopting cyclic pushover procedure. Laboratory, ATC-24, International Organization for Standardization (ISO), and Sequential Phased Displacement (SPD) protocols have been employed for investigation of effects of displacement histories on seismic demands. Comparison of results with nonlinear time history analysis suggest cyclic pushover analysis is of great utility in estimation of seismic displacement demands.

Kumar Pavan G.V.A et al (2015) have studied performance of ordinary and special moment resistant frames for compliance with Indian standard codes. Performance appraisal for various quake intensities and detailing bring out the importance of provision of strong column and weak beams.

2.4 INFLUENCE OF GEOMETRY AND MATERIAL MODELING ON PUSHOVER ANALYSIS RESULTS

Analysis results are greatly influenced by the approximations and simplifications. The success of pushover analysis in behavioral prediction heavily relies on how close the approximations and simplifications are to reality.

Utility of pushover analysis in performance appraisal has been elaborately discussed by Krawinkler and seneveritna (1998) by presenting strengths and weaknesses of the method. The importance of selection of appropriate load pattern has been highlighted and the efficacy of adaptive patterns have been illustrated.

Common pitfalls in pushover analysis have been reviewed by Naeim (1999) highlighting issues like loading pattern, P delta effects, shear failure mechanisms and detailing deficiencies and their influence on analysis results.

Tremendous amount of research has gone into study of parameters that affect analysis results, sensitivity of results to these parameters, uncertainties associated with results and to suggest means modes and methods to resolve issues and enhance capabilities of pushover analysis.

Paspuleti (2002) has modelled frame structure as a flexible model and a brittle model including all modeling parameters affecting pushover analysis results. Results indicate that displacements are more sensitive to modeling parameters. Uncertainty of pushover analysis methods to predict maximum roof displacement and inter storey drift has been addressed by Skokani (2002) with regards to welded steel moment frame buildings subjected to various levels of earthquake ground motion by comparing results with nonlinear time history analysis. The study revealed that a factor of 1.2 can be used to account for uncertainty in static nonlinear analysis.

To illustrate trends in the accuracy of various pushover load patterns, Inel Mehmet et al. (2003), have carried out pushover analysis by considering five load patterns, and the results obtained have been compared with dynamic analysis. A computer program that uses tabu search for weight minimization of two-dimensional framed structures by Kargahi Mohsen and Anderson James C. (2004) have been developed. From the analysis results it’s been observed that the search procedure was able to reduce the structural weight of the frame considered by 18.3% compared to the original design weight

An effective computer based technique that incorporates pushover analysis together with numerical optimization procedures to automate the pushover drift performance of design on a one bay one storey and ten storey two bay planar RC frame buildings has been presented by Zou. X-K and Chan. C-M (2005). Efficiency of steel reinforcement as a cost effective material for drift control beyond the occurrence of first yielding has been highlighted in this study.

Investigation of seismic behavior of concrete filled rectangular steel tube (CFRT) structures and RC structures have been performed by Jianguo Nie et al. (2006) to highlight the superiority of CFRT structures with respect to ductility and seismic performance.

Inel Mehmet and Ozmen Baytan Hayri (2006) have performed pushover analysis by considering user defined nonlinear hinge properties as well as default hinge properties as per ATC 40 and FEMA-356 guidelines to study the difference in the results. From the results it has been observed that displacement capacity of frames are considerably affected by plastic hinge length and transverse reinforcement spacing, while the same do not have any influence on base shear capacity.

To evaluate the performance of three framed buildings Kadid A and Boumrkik A (2008) have conducted nonlinear static pushover analysis and have observed that the plastic hinges are formed at the beam ends and column base of lower stories and then propagates to upper stories and further continues with yielding of interior immediate columns in the upper stories.

Ghodrati Amiri Gh.,et al., (2008) have aimed to use Genetic Algorithm in optimal design of reinforced concrete frames. In this work damage index as a design parameter has been used and the total weight of structure has been minimized which has led to decrease in construction cost. It has been suggested that it can also be a parameter to measure performance of structures as damage index predicts behavior of structure more accurately.

To predict and control the inelastic behavior under seismic loading and to determine the corresponding load factor, the design of SMRF has been studied using genetic algorithm by Kaveh.A and Dadfar.B (2008). From the study it has been observed that the design of SMRF for an arbitrary collapse mechanism and a value of ductility via members of constant cross sections is not always feasible and hence use of variable cross sections of beams have been proposed.

Peng Li and Weijian Yi (2008) have studied the effects of different axial load ratio and loading path on columns under cyclic loading. From the study it has been observed that axial load ratio and loading path affect plastic hinge length and as the axial load increases, plastic hinge length also increases. A procedure for incorporating structural modelling parameter uncertainties into probabilistic collapse risk assessments and other predictions of structural response has been proposed by Abbie,et al.(2009). Monte Carlo sampling with response surface methodology has been adopted. Uncertainties in structural components strength, stiffness, deformation capacities and cyclic deterioration have been considered for non-ductile and ductile frame structures of varying heights and results of the study indicate the methods utility in quantification of modeling uncertainties.

Balogh. T and Vigh. L.G (2012) have developed a numerical optimization algorithm and by expending the developed algorithm, results of structural optimization for various building cases have been presented. Through 18 illustrative examples, bracing systems with different level of energy dissipation (elastic concentric braced frame, dissipative concentric braced frame, buckling restrained brace frame) have been analyzed and guidelines have been provided for optimal structural configuration for economy in moderate seismicity regions.

Panandikar (Hede) Neena and Babu Narayan K.S. (2014) have investigated the sensitivity of pushover analysis results to geometric and material modeling parameters by comparing the analysis results with that of experimental investigations. Sensitivity of parameters like variations in material properties, inaccuracies in the placement of reinforcement, the effect of confinement of concrete and modeling techniques for elements and plastic hinges and their effects on pushover analysis results have been discussed.

2.5 RECENT ADVANCEMENTS IN PUSHOVER ANALYSIS

Upper-bound (UB) pushover analysis technique has been extended to unsymmetric-plan tall buildings to consider torsional effects into consideration by Mehdi P et al. (2015). Investigation suggest that responses are predictable to an affordable degree of accuracy.

Investigation by Alessandra F et al. (2016) on infilled frames by adopting double strut model to simulate the infill behavior, has indicated that the model likely arrests dangerous native shear failures that are sometimes neglected in pushover analysis.

A substitute method of estimating floor response spectra (FRS) on MDOF systems has been presented by Xiaolan Pan et al. (2016) considering numerous ‘generalized’ or ‘equivalent’ single degree of freedom (ESDOF) systems. This is an altered version of the previous MPA technique as it considers the contribution of the primary mode to yielding of upper modes when obtaining various ESDOF systems.

The energy-based pushover approach was 1st presented by Mountes Hernandez et al. in 2004 to cope with the distortions discovered in pushover curves of upper modes. Studies on this subject have also been performed by Tjhin T et al. (2005), Leelataviwat S et al. (2009), Jiang Y et al. (2010), Manoukas G et.al (2011), D’Ambrisi A et al. (2015) and Saedi Daryan A et al. (2017). Recently, this approach has been extended to asymmetric-plan buildings by Soleimani et al. (2017) referred to as E-MPA, in distinction to the standard approach that uses a roof displacement part as associate degree index to determine capability curves by Reyes JC et al. (2011) and Poursha M et al. (2011), E-MPA uses the work done by lateral loads and torques to provide capacity curves.

Soleimani S et al. (2018), have proposed an approximate two-component IDA technique on the idea of a bidirectional energy-based pushover (BEP) analysis by using the work done by lateral loads and torques through pushover analysis as an index to regulate the characteristics of the modal single-degree-of freedom systems. The accuracy of the proposed procedure has been verified on a two-way asymmetric 3-story building.

The performance of three numerical models with varying computational demand levels has been evaluated by Rafael A et al. (2018) and the accuracy of the calculated responses have been evaluated using experimental results. Model preparation and result acquisition times were found to involve a significant portion of the total computational demand of each model. An outline of the performance-based modeling procedures and the critical points for curtailing the computational demand while retaining the calculation accuracy have also been presented.

Fayaz R et al. (2018) have proposed a straightforward Dynamic-based Pushover analysis for plan asymmetric buildings (DPPA) to consider the consequences of torsional behavior as well as the higher modes in the applied lateral load pattern. The peak story drifts acquired from the response spectrum analysis (RSA) have been resolved into their translational and rotational components, and the associated equivalent static lateral forces and torsional moments can be determined. The method proposed has captured responses of shear building structures very accurately.

A modified spectrum-based pushover analysis (SPA) to consider the structural interaction between shear walls and frames and different damage modes of a dual wall-frame structure has been proposed by Yang L et al. (2018). The applicability and accuracy of MSPA in forecasting the seismic demand of dual wall-frame structures have been investigated through a case study of four 25-storey reinforced concrete wall-frame structures subjected to different levels of the input ground motions. Results have been compared with those obtained from nonlinear response history analysis. An extended spectrum based pushover analysis for predicting earthquake induced forces in tall buildings has also been suggested.

Zare Reza B et al. (2018) have investigated the pushover schemes for buildings with asymmetric plan and have proposed an extension of the energy-based adaptive pushover analysis (EAPA) procedure for the seismic design/assessment of 3D irregular structures and call the procedure energy-based pushover-analysis with torque-effects (EPT). The technique has well predicted the displacements and interstorey drifts of tall structures.

2.6 SUMMARY OF LITERATURE REVIEW

Review of available literature on pushover analysis presented in the preceding sections has elaborated the wide and varied range of research interests and efforts shown to enhance its utility in performance based design. Notwithstanding the efforts, unresolved issues exist and persist. The gap between analytical predictions and experimentally observed responses is being narrowed by refining and redefining geometry and material models.

2.7 OBJECTIVES OF THE PRESENT INVESTIGATION

Available literature manifests the wide and varying research interest in enhancement of prediction of performance. Sequence of hinge formation greatly influence pushover analysis results. Influence of sequence of plastic hinge formation on analysis results needs consideration to make behavior predictions more accurate. The present investigation is an attempt in this direction with the following objectives.

1. To study the effect of sequence of formation of plastic hinges on pushover analysis results.
2. Determination of bounds on displacement and base shear characteristics resulting from possible plastic hinge formation sequences.
3. Formulation of strategies to include uncertainties in plastic hinge formation sequences for refinement of pushover analysis results.

CHAPTER 3

EFFECT OF SEQUENCE OF PLASTIC HINGE FORMATION ON ANALYSIS RESULTS – A PROPPED CANTILEVER BEAM ILLUSTRATION

3.1 CHAPTER PROLOGUE

Load displacement characteristics are influenced by plastic hinge formation sequence. Though the collapse load is invariant, displacement is affected by the sequence of plastic hinge formation. This has been illustrated in the following sections considering the example of a propped cantilever beam carrying a central concentrated load.

3.2 MODELLING DETAILS

A propped cantilever beam AB of Span **'$\boldsymbol{l}$' = 2m** loaded at mid span is considered for the analysis is as shown in the fig 3.1. The section considered is ISMB 300 @ 46.1 kg/m which has the following properties.

Sectional modulus	Z_X	= 599 cm^3
Plastic Section modulus	Z_P	= 683 cm^3
Shape Factor		= 1.14
Elastic modulus,	E	= 200 GPa
Yield moment	M_Y	= 149.75 kN-m
Plastic moment	M_P	= 170.72 kN-m

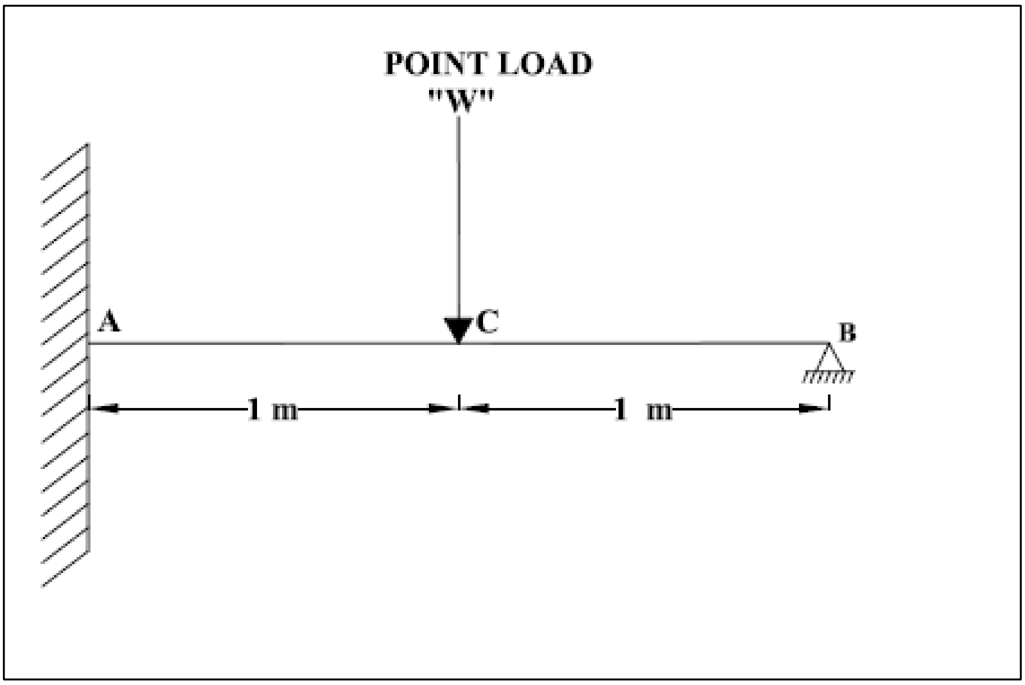


Fig. 3.1. Propped cantilever loaded at mid span

Case 1 – Hinge formation sequence 'A-C'

For propped cantilever beam under consideration two plastic hinges are necessary for collapse mechanism, and these form one at fixed end and other at the center of the beam (self weight of beam is ignored).

In case 1, the plastic hinge formation sequence is 'A-C' that is first plastic hinge forms at the fixed support and the second plastic hinge forms at the center of the beam.

To show the change in displacement characteristics due to change in plastic hinge formation sequence the propped cantilever beam in case 1 is given an upward displacement of 2mm at the propped end (by considering construction defects) as shown in fig 3.2.

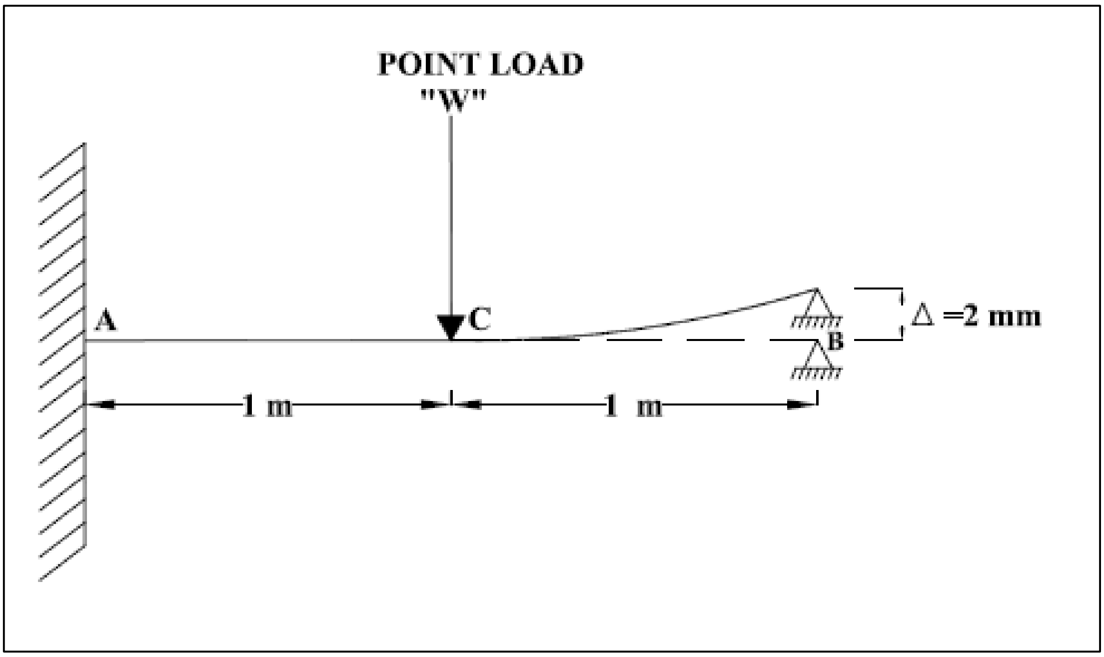


Fig 3.2. Propped cantilever with upward displacement of 2mm at point B

Case 2 – Hinge formation sequence 'C–A'

For case 2, first plastic hinge forms at 'C' and plastic hinge 'A' forms subsequently.

For both the cases bending moment diagram at collapse is the same and is as shown in fig 3.3.

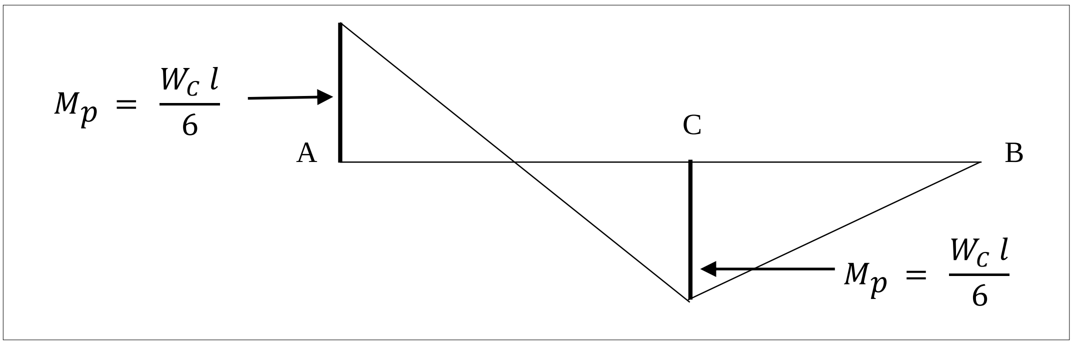


Fig. 3.3. Bending moment diagram at collapse

From bending moment diagram, collapse load can be determined.

$$W_C = \frac{6Mp}{l} = 512.145 \text{ kN} \qquad \text{(i)}$$

3.3. MOMENT - DISPLACEMENT EQUATIONS

Equations for Moments and displacements considered in this analysis (i.e. for case 1 and case 2) obtained from slope deflection method are presented below.

For Case 1

Moment at 'A', $$M_A = -\frac{3Wl}{16} \qquad (1)$$

Moment at 'C', $$M_C = \frac{5Wl}{32} \qquad (2)$$

Displacement at 'C' till first plastic hinge forms at 'A',

$$\delta_C = \frac{7Wl^3}{798EI} \qquad (3)$$

Displacement at 'C' after formation of first plastic hinge at 'A'

		Displacement		Displacement due		Displacement
δ_{AC}	$=$	at 'C'	$+$	to additional load	$+$	due to
		(δ_C)		(W_A)		rotation (θ_A)

For Case 2

Moment at 'A', $$M_A = -\frac{3Wl}{16} - \frac{103.05}{l} \qquad (5)$$

Moment at 'C', $$M_C = \frac{5Wl}{32} + \frac{51.52}{l} \qquad (6)$$

Displacement at 'C' till first plastic hinge forms at 'C'

$$\delta_C = \frac{7Wl^3}{798EI} \tag{7}$$

Displacement at 'C' after formation of first plastic hinge at 'C' (8)

$$\delta_{CC} = \begin{array}{c}\text{Displacement}\\ \text{at 'C'}\\ (\delta_C)\end{array} + \begin{array}{c}\text{Displacement due}\\ \text{to additional load}\\ (W_A)\end{array} + \begin{array}{c}\text{Displacement}\\ \text{due to}\\ \text{rotation}\\ (\theta_C)\end{array}$$

In the above equations 'W' is point load, 'l' is span of the beam, E is elastic modulus and I is moment of inertia of the beam considered.

The corresponding moments and displacements have been determined for incremental loads for both cases are listed in table 3.1.

Table 3.1. Moments and displacements for propped cantilever beam

Case (1) Propped cantilever loaded at mid span				Case(2) Propped cantilever with an upward displacement at propped end		
Load in ‘kN’	**Moments in ‘kN-m’**		**Max Displacement Δ in ‘mm’**	**Moments in ‘kN-m’**		**Max Displacement δ in ‘mm’**
	at ‘A’	**at ‘C’**		**at ‘A’**	**at ‘C’**	
250	93.75	78.125	1.06	42.13	103.94	1.06
400	150	125	1.69	98.38	150.81	1.69
410	153.75	128.125	1.74	102.13	153.94	1.74
420	157.5	131.25	1.78	105.88	157.06	1.78
430	161.25	134.375	1.82	107.76	158.01	1.82
440	165	137.5	1.86	109.63	160.19	1.86
450	168.75	140.625	1.91	113.38	163.31	1.91
455.24	170.72	142.2625	1.93	117.13	166.44	1.93
460		144.6425	2.04	120.88	169.56	1.95
463.7		146.220	2.11	122.27	170.72	2.15
470		149.6425	2.29	128.57		2.75
480		154.6425	2.55	138.57		3.24
490		159.6425	2.82	148.57		3.72
500		164.6425	3.12	158.57		4.21
510		169.6425	3.48	168.57		4.29
512.145		170.72	3.58	170.72		4.32

3.4 RESULTS AND DISCUSSIONS

For propped cantilever beam in case 1, first plastic hinge forms at the fixed support at a load of 455.24 kN with corresponding displacement of 1.93mm. Once the first plastic hinge forms at the fixed support, the beam will become determinate and behaves as a simply supported beam as the fixed support is free to rotate now. From this stage, to the central displacement, displacement due to additional load and rotation of plastic hinge get added and hence total displacement is obtained.

Addition of load beyond formation of first plastic hinge is possible till the second plastic hinge forms at the center. At collapse for case 1, maximum displacement obtained is 3.58mm.

For case 2, the first plastic hinge forms at center as moment is numerically maximum here in the elastic behavior range. The second plastic hinge forms later at the fixed support. The load and corresponding displacement when first plastic hinge forms are 463.7kN and 2.15mm respectively. The maximum displacement at collapse in this case is 4.32mm. Load vs. displacement curves for both the cases are as shown in fig 3.4.

It is very interesting to note that the load and corresponding displacements are different at the instant of formation of first plastic hinge. The maximum displacement at incipient failure are also different, but the collapse load is the same.

The Second case shows 21% increase in displacement at collapse as compared to first case. This is because, the cantilever in which plastic hinge forms first at the midspan becomes more flexible as it behaves as an assemblage of cantilever on one half and suspended beam on the other.

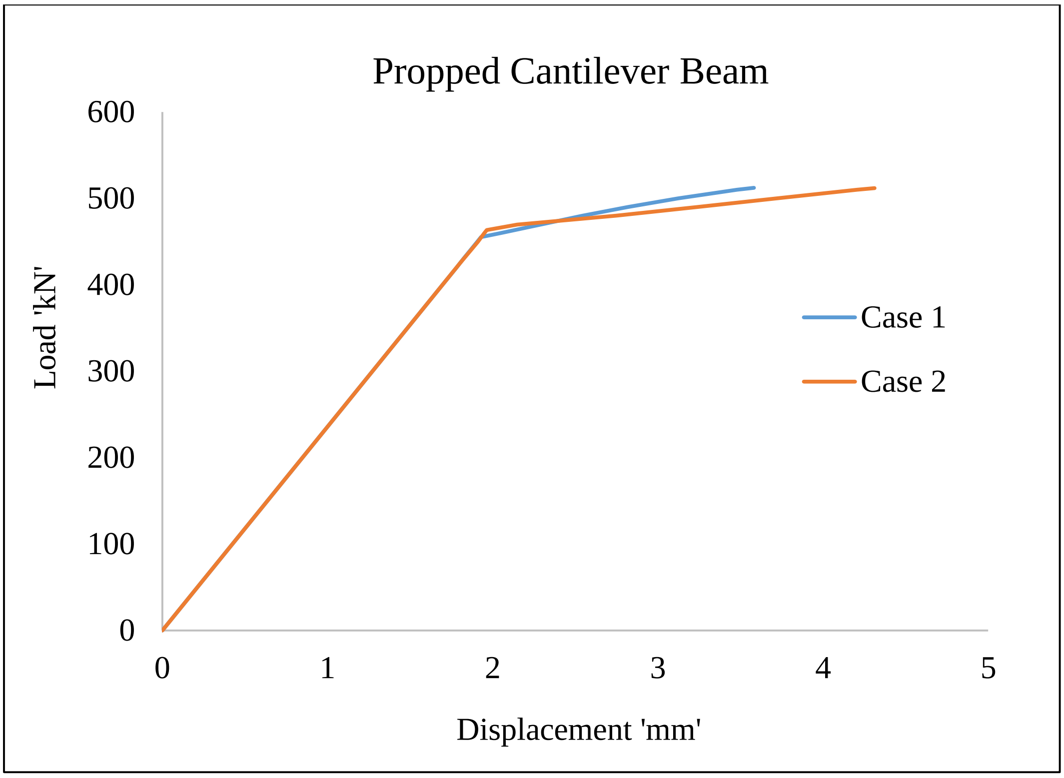


Fig. 3.4. Load vs. displacement of a propped cantilever beams

CHAPTER 4

DETERMINATION OF BOUNDS ON PERFORMANCE CHARACTERISTICS – SINGLE BAY SINGLE STOREY RC FRAME EXAMPLE

4.1 CHAPTER PROLOGUE

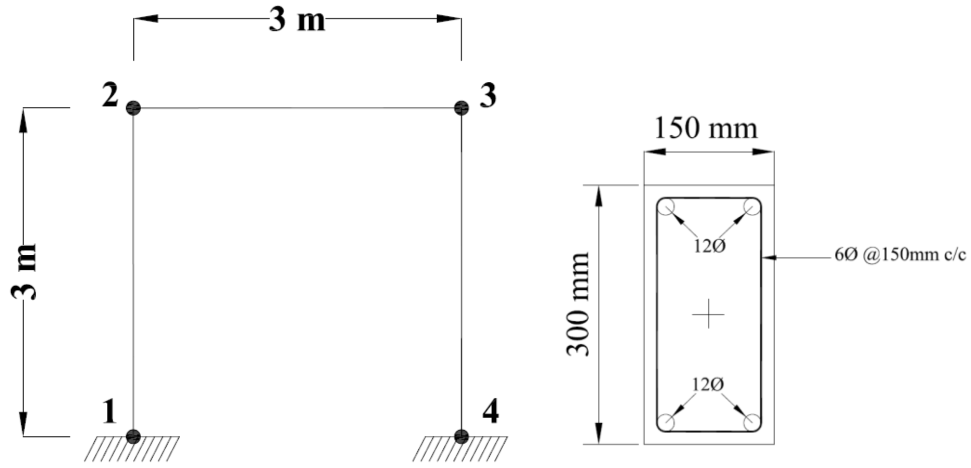
Influence of sequence of plastic hinge formation on load displacement behavior has been illustrated with the aid of propped cantilever beam in chapter 3. Pushover analysis results, analysis being a step by step procedure where plastic hinges form leading the structure to a mechanism should necessarily be influenced by sequence of formation of plastic hinge.

It is usual in analysis to model the geometry and materials assuming that no uncertainties are associated. Deviations from strength and geometry assumed in analysis can lead to a plastic hinge formation sequence other than the one for which analysis is made.

In this chapter a single bay single storey RC frame has been considered for pushover analysis allowing variations in strength and detailing parameters to investigate their effect on plastic hinge formation sequence and the subsequent influence on analysis results.

4.2 MODELING DETAILS

An RC frame of height 3m and width 3m with beam and column dimensions as shown in fig 4.1 considered for analysis is modelled in SAP2000.



(a) Frame elevation (b) C/S of elements

Fig. 4.1. Frame and element details

In this study, moment hinge M3 is assigned at member ends where flexural yielding is assumed to occur for both beams and columns, and has a moment curvature relationship as shown in fig 4.2. Nonlinear static pushover cases are defined and displacement controlled analysis is carried out. The pushover results along with pushover curves obtained at the end of nonlinear analysis is captured and the sequence of plastic hinge formation is tracked.

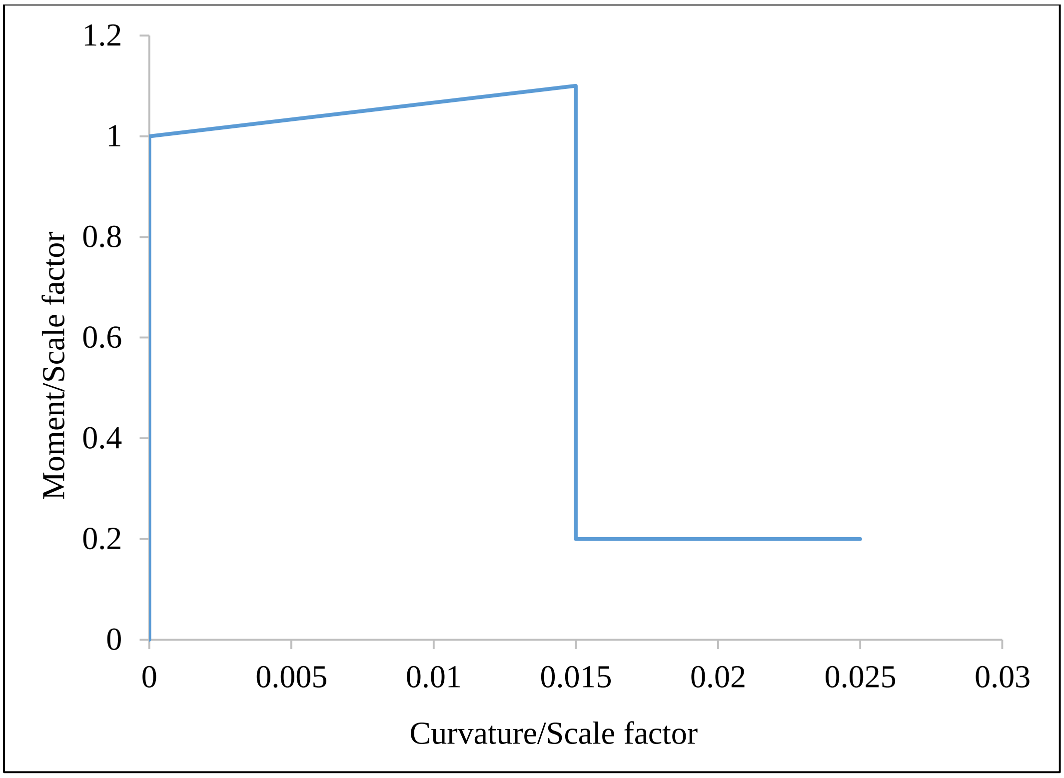


Fig. 4.2. Moment curvature relationship of RC element

4.3 INFLUENCE OF MATERIAL STRENGTH AND EFFECTIVE COVER VARIATIONS ON PUSHOVER ANALYSIS RESULTS

Parametric studies have been carried out for variability in material strengths and effective covers assigning discrete values within the variation range and also allowing values to randomly vary within the range.

4.3.1. Pushover analysis results for material strength and effective covers – Discrete values assigned within variation range assumed

Influence of material strengths and effective cover variations have been studied by considering five values for steel strength (fs) (350,380,415,445,475 MPa), five values of concrete strength (fc) (17, 18.5, 20, 21.5 and 23 MPa) and effective cover (dc) (25, 27.5, 30, 32.5 and 35) by allowing ± 15% variation from central values. Maximum base shear and corresponding displacement values from pushover analysis obtained are reported in tables 4.1 – 4.5.

Also pushover curves for various effective covers and for varying concrete and steel strengths are presented in figs 4.3 to 4.12.

Table 4.1. Maximum base shears and corresponding displacements for varying concrete strength (fc ‘MPa’) and steel strength (fs ‘MPa’) for effective cover of 25mm

fc / fs	17		18.5		20		21.5		23	
	Disp ‘m’	**Base Shear 'kN’**	**Disp ‘m’**	**Base Shear ‘kN’**	**Disp ‘m’**	**Base Shear ‘kN’**	**Disp ‘m’**	**Base Shear ‘kN’**	**Disp ‘m’**	**Base Shear ‘kN'**
350	0.018716	43.294	0.018507	43.384	0.018323	43.467	0.018069	43.526	0.018008	43.619
380	0.019138	46.937	0.018941	47.078	0.018741	47.171	0.018561	47.259	0.018399	47.342
415	0.02085	51.006	0.01943	51.223	0.019227	51.461	0.019033	51.579	0.018856	51.672
445	0.021914	54.378	0.021388	54.632	0.019917	54.835	0.019413	55.058	0.019238	55.289
475	0.023401	58.079	0.022947	58.122	0.021336	58.227	0.019987	58.464	0.019536	58.732

Table 4.2. Maximum base shears and corresponding displacements for varying concrete strength (fc 'MPa') and steel strength (fs 'MPa') for effective cover of 27.5mm

fs \ fc	17		18.5		20		21.5		23	
	Disp 'm'	**Base Shear 'kN'**	**Disp 'm'**	**Base Shear 'kN'**	**Disp 'm'**	**Base Shear 'kN'**	**Disp 'm'**	**Base Shear 'kN'**	**Disp 'm'**	**Base Shear 'kN'**
350	0.018629	42.977	0.018422	43.072	0.018239	43.16	0.018131	43.137	0.017928	43.364
380	0.019893	46.563	0.018854	46.724	0.018515	46.729	0.018478	46.972	0.018313	47.06
415	0.020385	50.546	0.019843	50.75	0.019104	50.948	0.018947	51.186	0.018767	51.296
445	0.021918	54.025	0.020818	54.127	0.019932	54.354	0.019323	54.577	0.019048	54.666
475	0.023401	57.563	0.021996	57.712	0.020901	57.846	0.019774	57.975	0.019542	58.21

Table 4.3. Maximum base shear and corresponding displacements for varying concrete strength (fc 'MPa') and steel strength (fs 'MPa') for effective cover of 30mm

fs \ fc	17		18.5		20		21.5		23	
	Disp 'm'	**Base Shear 'kN'**	**Disp 'm'**	**Base Shear 'kN'**	**Disp 'm'**	**Base Shear 'kN'**	**Disp 'm'**	**Base Shear 'kN'**	**Disp 'm'**	**Base Shear 'kN'**
350	0.018544	42.695	0.018336	42.843	0.01814	42.933	0.018093	42.823	0.017949	42.842
380	0.019015	46.248	0.018779	46.387	0.018569	46.523	0.018471	46.433	0.018319	46.58
415	0.020154	50.142	0.019255	50.358	0.019055	50.594	0.018887	50.244	0.018732	50.499
445	0.021943	53.591	0.019984	53.661	0.019509	53.922	0.019248	53.551	0.01908	53.793
475	0.023511	57.111	0.021013	57.265	0.020742	57.416	0.019542	56.893	0.019632	57.117

Table 4.4. Maximum base shear and corresponding displacements for varying concrete strength (fc 'MPa') and steel strength (fs 'MPa') for effective cover of 32.5mm

fc / fs	17		18.5		20		21.5		23	
	Disp 'm'	**Base Shear 'kN'**	**Disp 'm'**	**Base Shear 'kN'**	**Disp 'm'**	**Base Shear 'kN'**	**Disp 'm'**	**Base Shear 'kN'**	**Disp 'm'**	**Base Shear 'kN'**
350	0.018449	41.851	0.01824	42.045	0.018133	42.489	0.018043	42.58	0.017857	42.67
380	0.018191	44.999	0.018398	44.992	0.018561	45.834	0.018406	46.104	0.018212	46.106
415	0.01994	48.883	0.019142	49.157	0.018988	49.645	0.018816	49.897	0.018638	49.908
445	0.021041	52.248	0.019897	52.485	0.019436	52.947	0.019215	53.053	0.018983	53.199
475	0.023944	55.699	0.020967	55.849	0.019815	56.26	0.019584	56.514	0.019332	56.521

Table 4.5. Maximum base shear and corresponding displacements for varying concrete strength (fc 'MPa') and steel strength (fs 'MPa') for effective cover of 35mm

fc / fs	17		18.5		20		21.5		23	
	Disp 'm'	**Base Shear 'kN'**	**Disp 'm'**	**Base Shear 'kN'**	**Disp 'm'**	**Base Shear 'kN'**	**Disp 'm'**	**Base Shear 'kN'**	**Disp 'm'**	**Base Shear 'kN'**
350	0.018417	41.451	0.018193	41.755	0.018	41.863	0.017856	42.182	0.017708	42.398
380	0.0188	44.713	0.018593	44.979	0.018438	45.181	0.01824	45.431	0.018108	45.813
415	0.019436	48.478	0.019427	48.73	0.01884	48.959	0.018686	49.215	0.018543	49.475
445	0.021373	51.807	0.020969	51.941	0.019284	52.188	0.019184	52.506	0.019027	52.759
475	0.023887	55.236	0.02102	55.414	0.020534	55.617	0.019961	55.777	0.019634	55.999

4.3.1.1 Results and discussions

From table 4.1 it can be seen that for effective cover of 25mm and concrete strength of 17MPa, the displacement varies from 0.018716m to 0.023401m with ± 15% variation in steel strength and the corresponding base shears are 43.294kN and 58.079kN respectively. It can be observed from the tables that both base shear and displacement values considerably vary with variations in steel strength. Variations in analysis values are not so sensitive to variations in concrete strength. This is due to the fact that all elements of the frame investigated are under reinforced.

The variability of analysis results with variations in effective cover is insignificant i.e. maximum displacement and base shear observed for 25mm effective cover are 0.023401m and 58.079kN respectively and those of 35mm cover are 0.023887m and 55.23kN respectively.

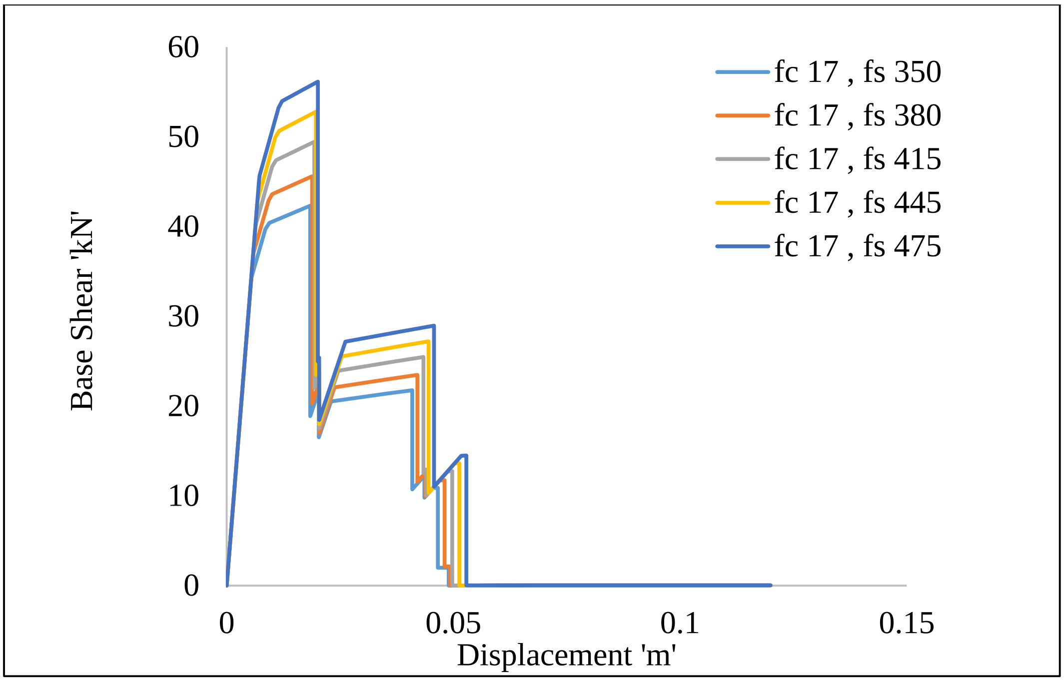


Fig. 4.3. Pushover curves for effective cover 25mm and steel strengths varying

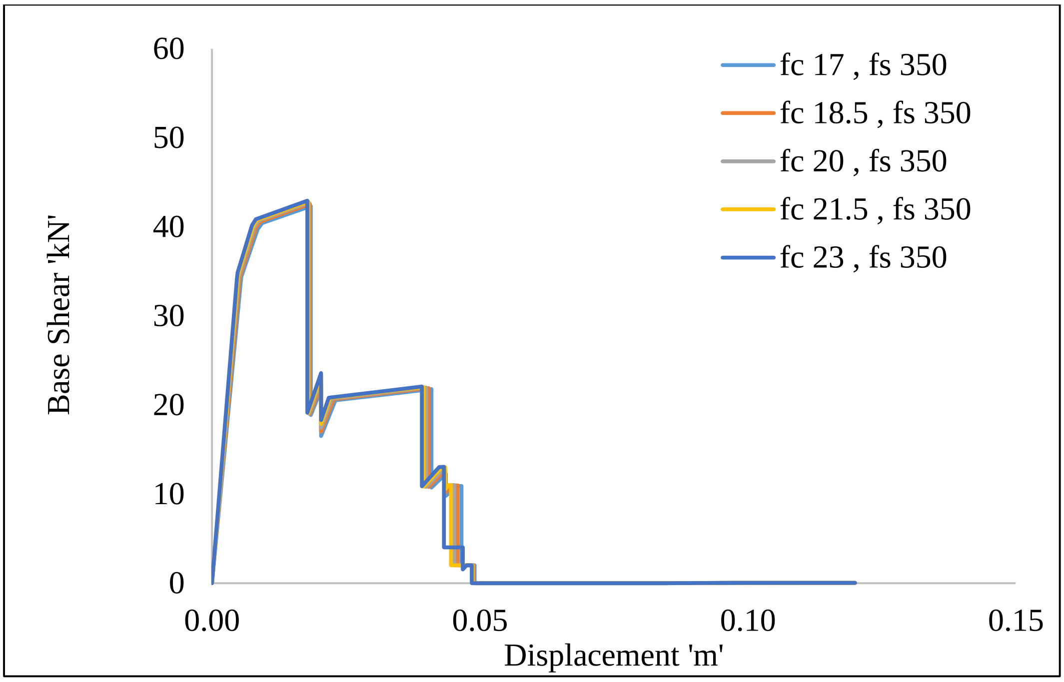


Fig. 4.4. Pushover curves for effective cover 25mm and concrete strengths varying

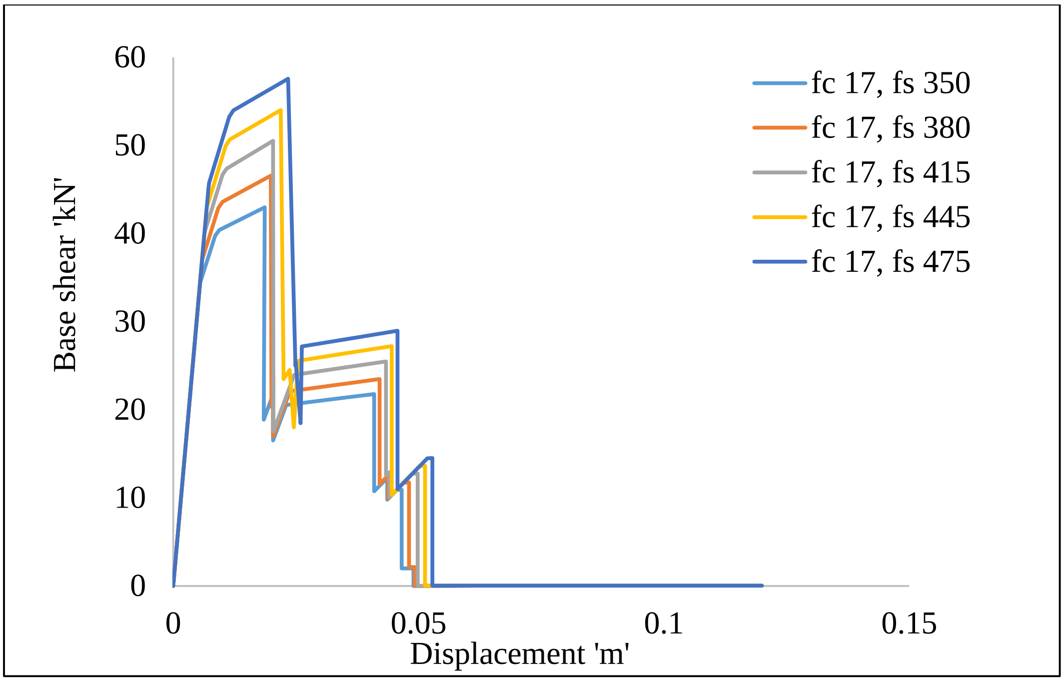


Fig. 4.5. Pushover curves for effective cover 27.5mm and steel strengths varying

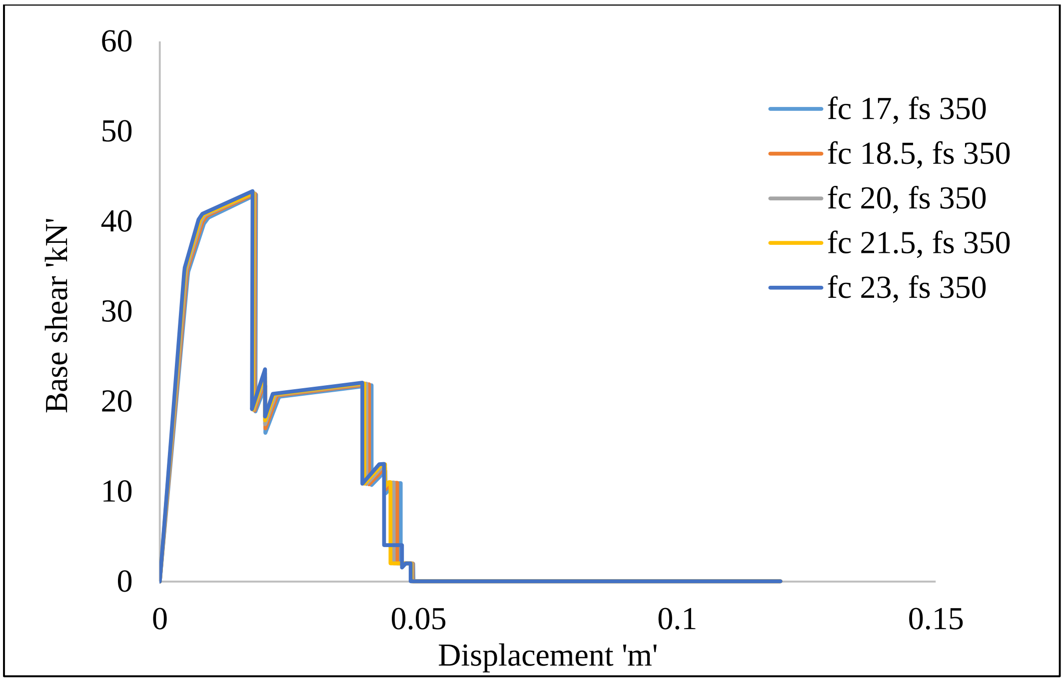


Fig. 4.6. Pushover curves for effective cover 27.5mm and concrete strengths varying

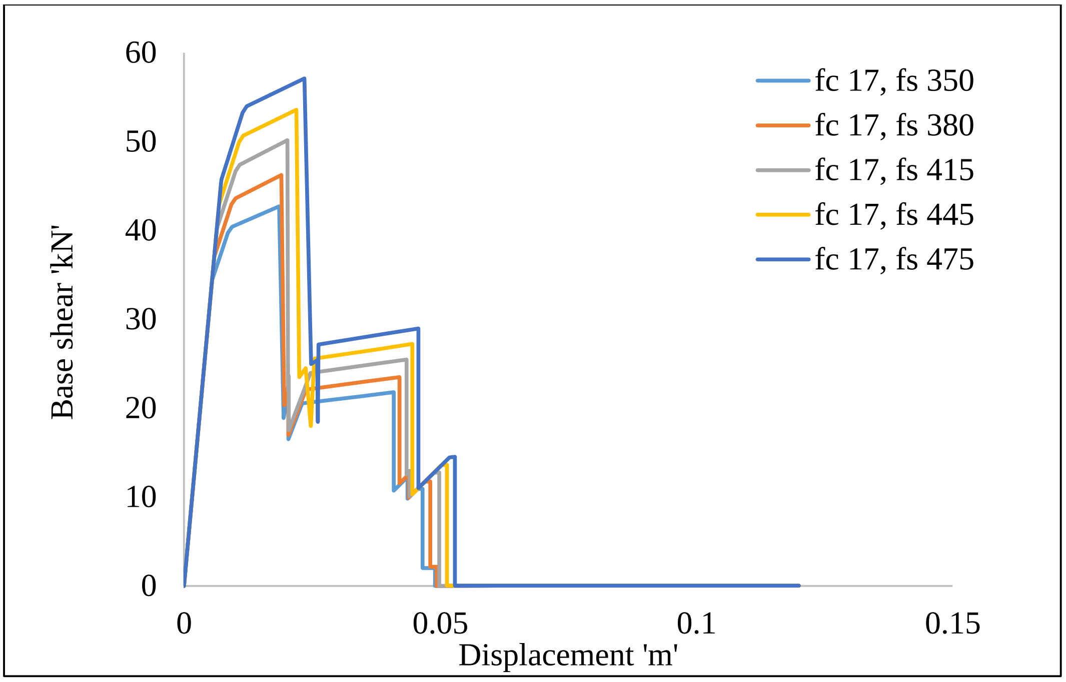


Fig. 4.7. Pushover curves for effective cover 30mm and steel strengths varying

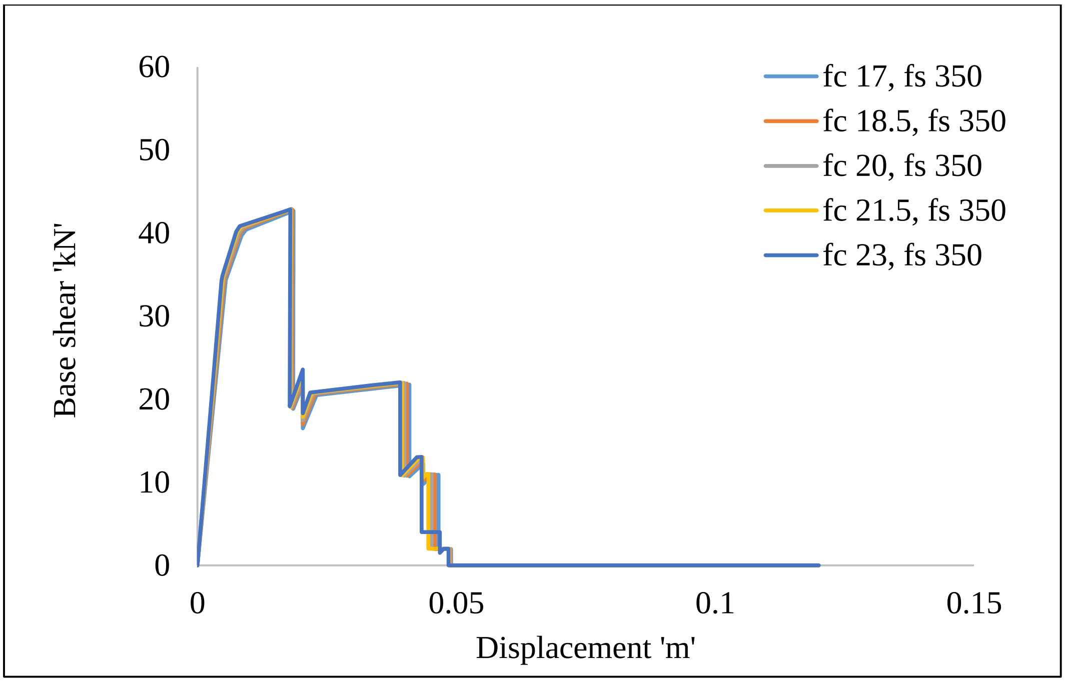


Fig. 4.8. Pushover curves for effective cover 30mm and concrete strengths varying

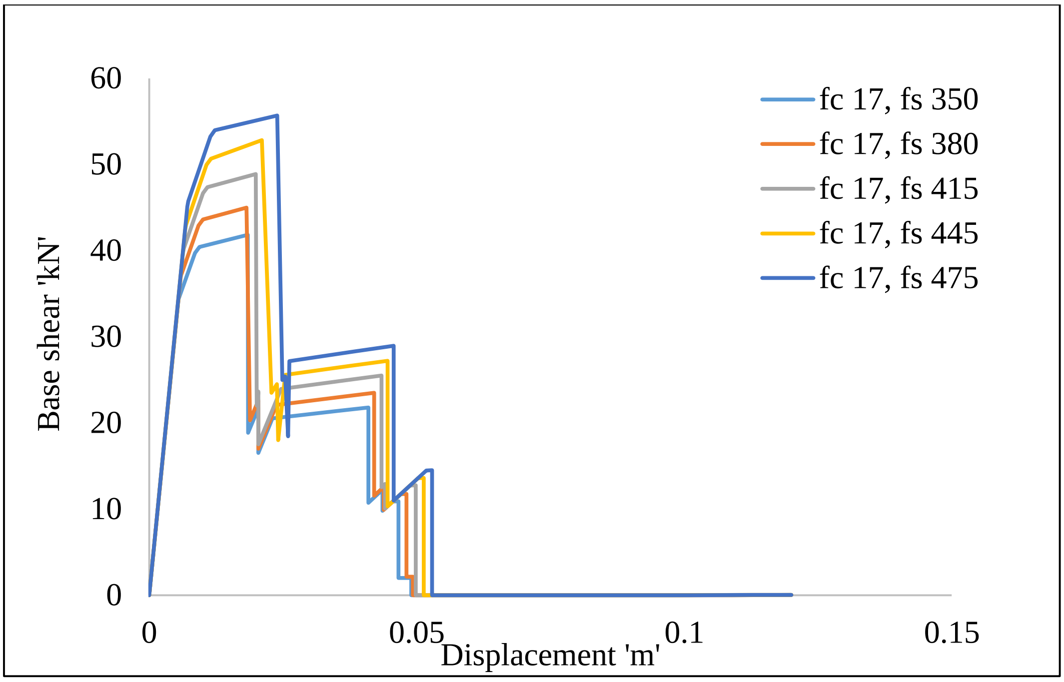


Fig. 4.9. Pushover curves for effective cover 32.5mm and steel strengths varying

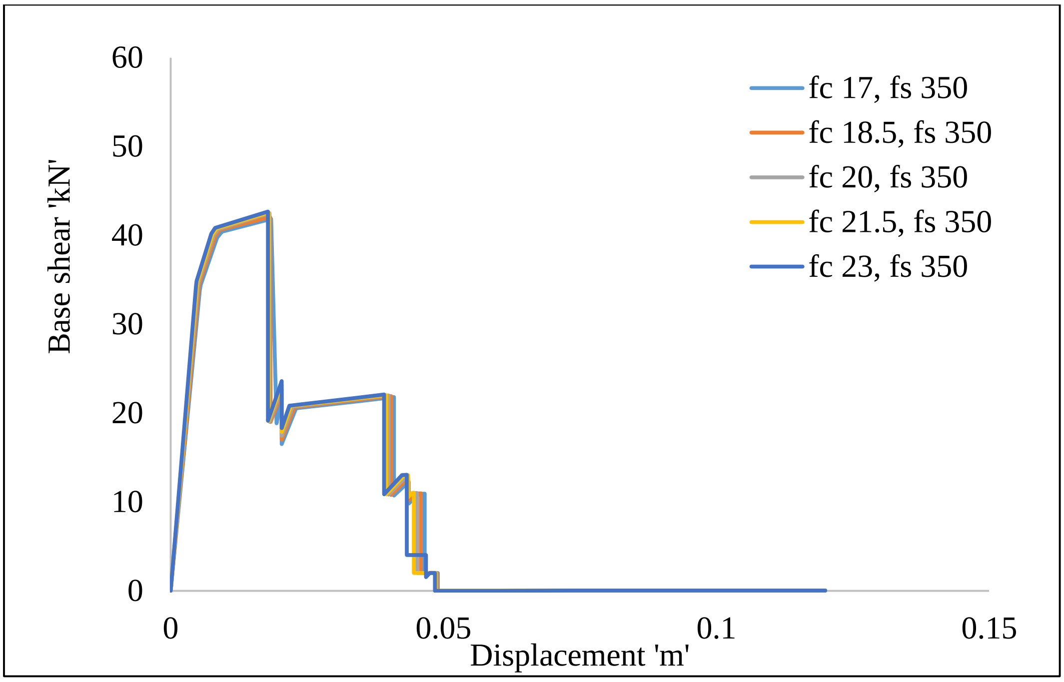


Fig. 4.10. Pushover curves for effective cover 32.5mm and concrete strengths varying

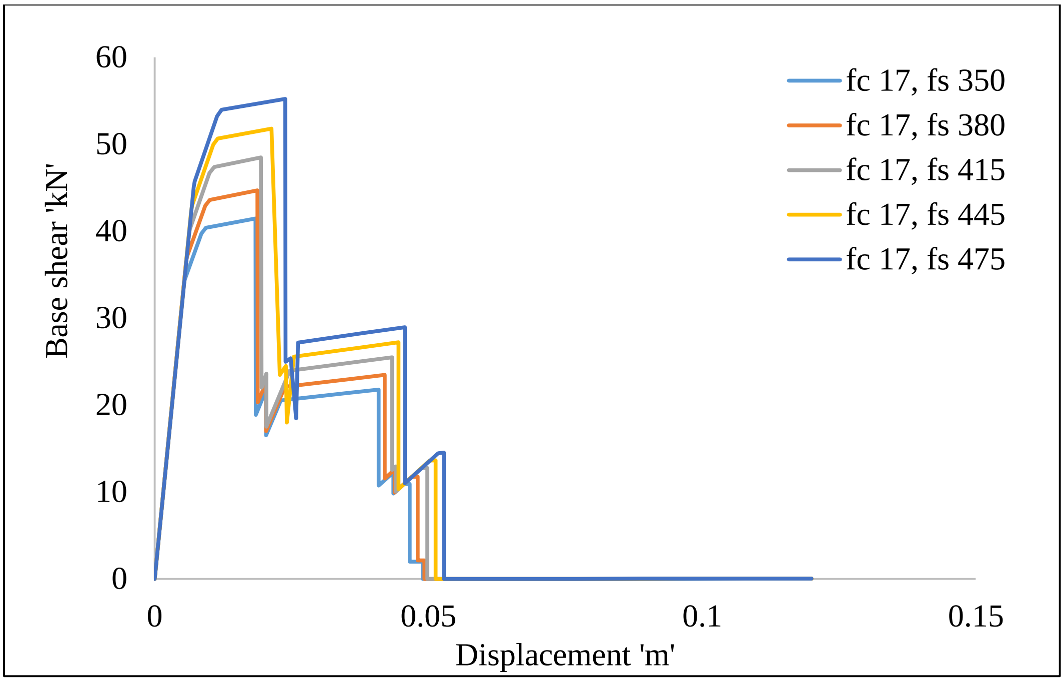


Fig. 4.11. Pushover curves for effective cover 35mm and steel strengths varying

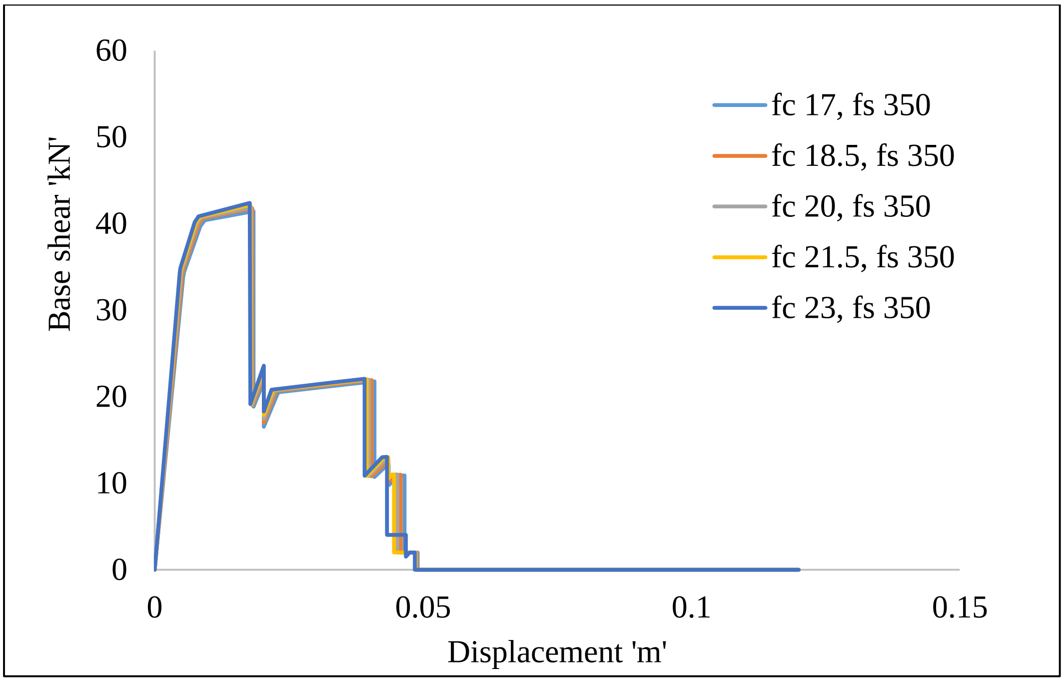


Fig. 4.12. Pushover curves for effective cover 35mm and concrete strengths varying

4.3.2. Pushover analysis results for material strength and effective covers – allowed random variations within prescribed range of ± 15%

In this parametric study pushover analysis has been performed on the same frame for randomized values over a preselected range of variations in steel strengths (fs) from 350 to 475MPa, concrete strength (fc) from 17 to 23MPa and effective cover (dc) from 25mm to 35mm. Base shear and displacements obtained for 101 random samples are presented in table 4.6. The following equations are used to generate random values between specified ranges of steel strengths (fs), concrete strength (fc) and effective cover (dc).

Steel strength (fs) = Rand ( ) × (475 – 350) + 350 (1)

Concrete strength (fc) = Rand ( ) × (23 – 17) + 17 (2)

Effective Cover (dc) = Rand ( ) × (35 – 25) + 35 (3)

Table 4.6. Pushover analysis results for random values of steel strength fs, concrete strength fc and effective cover dc

Sl No.	fs	fc	dc	Base Shear ‘kN’	Displacement ‘m’
1	462.47	18.66	26.37	54.76	0.01964
2	405.68	19.00	30.16	47.999	0.018737
3	444.33	18.04	30.01	52.098	0.019355
4	376.15	18.62	32.82	44.397	0.01833
5	448.79	17.91	31.57	52.326	0.019365
6	386.82	18.91	31.50	45.764	0.018472
7	468.16	18.71	34.95	54.023	0.019337
8	463.09	21.42	26.19	55.296	0.019282
9	399.43	21.27	33.52	47.272	0.018283
10	449.46	17.21	32.03	52.216	0.019465
11	367.32	22.25	34.60	43.914	0.017806
12	436.52	19.63	29.13	51.631	0.019072

13	390.47	21.81	28.65	47.063	0.018312
14	471.78	22.98	31.06	55.744	0.019005
15	425.02	19.71	28.21	50.464	0.018959
16	421.30	21.91	32.75	49.835	0.01849
17	401.66	17.73	25.90	47.965	0.019013
18	389.16	17.89	30.07	46.032	0.018681
19	358.13	22.62	30.86	43.314	0.017768
20	437.97	21.15	29.70	51.953	0.018882
21	471.09	17.64	26.37	55.572	0.019916
22	432.93	20.97	26.45	51.845	0.018968
23	367.54	22.29	28.94	44.567	0.017982
24	410.78	22.64	27.91	49.456	0.018486
25	436.92	22.95	25.05	52.86	0.018849
26	396.11	20.81	27.87	47.558	0.018504
27	378.45	18.43	33.85	44.47	0.018337
28	432.71	19.15	29.46	51.084	0.019076
29	431.88	21.87	29.88	51.382	0.018726
30	452.21	19.37	31.32	52.977	0.019207
31	411.66	18.43	27.38	48.93	0.018991
32	462.11	18.91	32.27	53.842	0.019352
33	411.40	21.72	29.19	49.231	0.018536
34	465.65	21.06	26.23	55.516	0.019355
35	357.30	20.91	28.93	43.239	0.017994
36	364.65	22.54	33.11	43.808	0.017792
37	397.18	21.32	29.28	47.611	0.018413
38	424.52	22.29	26.88	51.066	0.018711
39	365.01	21.87	27.25	44.391	0.018042
40	361.12	18.12	30.49	42.992	0.018293
41	452.97	17.06	25.98	53.532	0.019789
42	454.77	22.96	30.06	54.032	0.018859
43	421.73	21.08	33.69	49.61	0.018544

44	366.22	17.25	29.06	43.573	0.018513
45	397.94	17.45	28.69	47.107	0.018903
46	351.71	21.84	32.16	42.348	0.017723
47	470.14	17.51	34.72	54.061	0.01956
48	371.16	21.93	28.99	44.947	0.018069
49	474.86	18.10	34.44	54.715	0.019536
50	459.38	22.41	31.20	54.274	0.018923
51	410.93	17.64	26.78	48.817	0.019113
52	437.86	19.50	34.37	50.93	0.018889
53	468.98	17.36	34.96	53.877	0.01956
54	415.64	21.61	29.89	49.57	0.018569
55	389.48	22.02	32.85	46.431	0.018128
56	373.50	20.86	25.70	45.479	0.018309
57	369.11	22.77	28.78	44.832	0.017959
58	413.43	18.89	27.34	49.207	0.018954
59	365.71	22.15	26.96	44.536	0.018028
60	406.25	19.81	30.11	48.213	0.01865
61	413.31	17.43	25.05	49.346	0.019238
62	427.83	19.97	27.92	50.864	0.018972
63	428.73	22.55	27.59	51.445	0.018706
64	362.20	20.31	28.65	43.757	0.018136
65	467.29	19.30	26.07	55.457	0.019619
66	456.50	20.43	30.87	53.687	0.019138
67	459.32	19.03	28.80	54.099	0.019447
68	463.63	21.35	32.26	54.406	0.019048
69	422.64	17.76	28.82	49.847	0.019168
70	414.00	18.06	30.90	48.645	0.018935
71	357.27	17.24	28.04	42.702	0.018437
72	439.22	21.56	34.37	51.44	0.018659
73	427.07	18.34	27.31	50.622	0.0192
74	359.18	18.82	25.51	43.59	0.018356

75	463.07	19.15	27.39	54.718	0.019535
76	351.13	17.85	27.77	42.194	0.018295
77	470.99	19.02	34.95	54.385	0.019325
78	435.89	17.49	32.47	50.711	0.019228
79	450.21	19.38	34.52	52.235	0.019045
80	375.69	18.52	33.83	44.2	0.018297
81	380.88	18.80	27.98	45.54	0.018542
82	464.18	19.80	31.34	54.356	0.019292
83	371.56	19.76	26.51	44.939	0.018372
84	386.90	19.15	28.62	46.21	0.018551
85	353.61	22.41	34.93	42.307	0.017605
86	414.59	20.91	26.92	49.743	0.018742
87	428.48	18.80	26.01	51.079	0.019203
88	426.18	21.95	28.04	50.99	0.018723
89	355.53	17.73	29.97	42.388	0.018289
90	396.78	18.21	34.84	46.274	0.018546
91	429.92	19.27	28.87	50.887	0.019049
92	464.22	21.52	34.46	54.138	0.01894
93	359.66	18.93	27.93	43.285	0.018273
94	420.10	19.65	34.60	49.006	0.018653
95	371.93	17.64	34.65	43.536	0.018326
96	358.15	20.19	28.94	43.256	0.018089
97	370.83	17.88	34.87	43.436	0.018275
98	364.66	17.11	29.27	43.351	0.018504
99	428.36	20.24	34.06	50.13	0.0187
100	399.81	22.35	33.36	47.526	0.018189
101	458.38	20.19	30.17	53.963	0.01922

4.3.2.1 Results and discussions

From analysis, base shear and displacement considering random values for strength of concrete (fc), strength of steel (fs) and effective cover (dc) for a single storey single bay RC frame gives mean value of base shear as 49.022 kN with standard deviation of 4.103 and mean value of displacement obtained is 0.01874m with standard deviation of 0.0005129.

Confidence interval (CI) at a level of significance of 0.05 is estimated. The results obtained are as shown in table 4.7.

Table 4.7. Statistical analysis results for random values of steel strengths fs, concrete strengths fc and effective covers dc

Output	Mean	Standard deviation	C.O.V '%'	95% Confidence Interval (CI)
Base Shear 'kN'	49.022	4.103	8%	(40.98, 57.06)
Displacement 'm'	0.0187	0.0005129	2%	(0.0177, 0.0197)

The coefficient of variation obtained for base shear and displacement are 8% and 2%, indicating variations in material strengths and cover to steel reinforcement influences base shear more than displacements. Histogram for base shear and displacement are presented in fig 4.5 and 4.6.

Closest results obtained from case 2 i.e. by allowing random variations within prescribed range of ± 15% have been compared with those obtained by allowing discrete values within variation range assumed are presented in table 4.8.

Table. 4.8. Comparison of results for variables randomized within prescribed range of ± 15% with results from variables assigned discrete values within variability range

Parameter Considered	Strength of concrete fc 'MPa'	Strength of steel fs 'MPa'	Effective cover dc 'mm'	Maximum base shear 'kN'	Corresponding displacement 'm'
Case 1	17	415	25	51.006	0.02085
Case 2	17.43	413.31	25.05	49.346	0.019238
Case 1	18.5	475	35	55.414	0.02102
Case 2	18.10	474.86	34.44	54.715	0.019536

Comparison of results provided in table 4.8 suggests computational time and efforts can be reduced by randomizing variables.

± 15% variations in material strength and detailing have resulted in an increase in displacement of 13%. Whereas, in illustration of a propped cantilever beam, plastic hinge sequence change alone has affected in 21% increase in displacement, indicating changes in plastic hinge formation sequence has more influence on pushover analysis results than variations in material strengths.

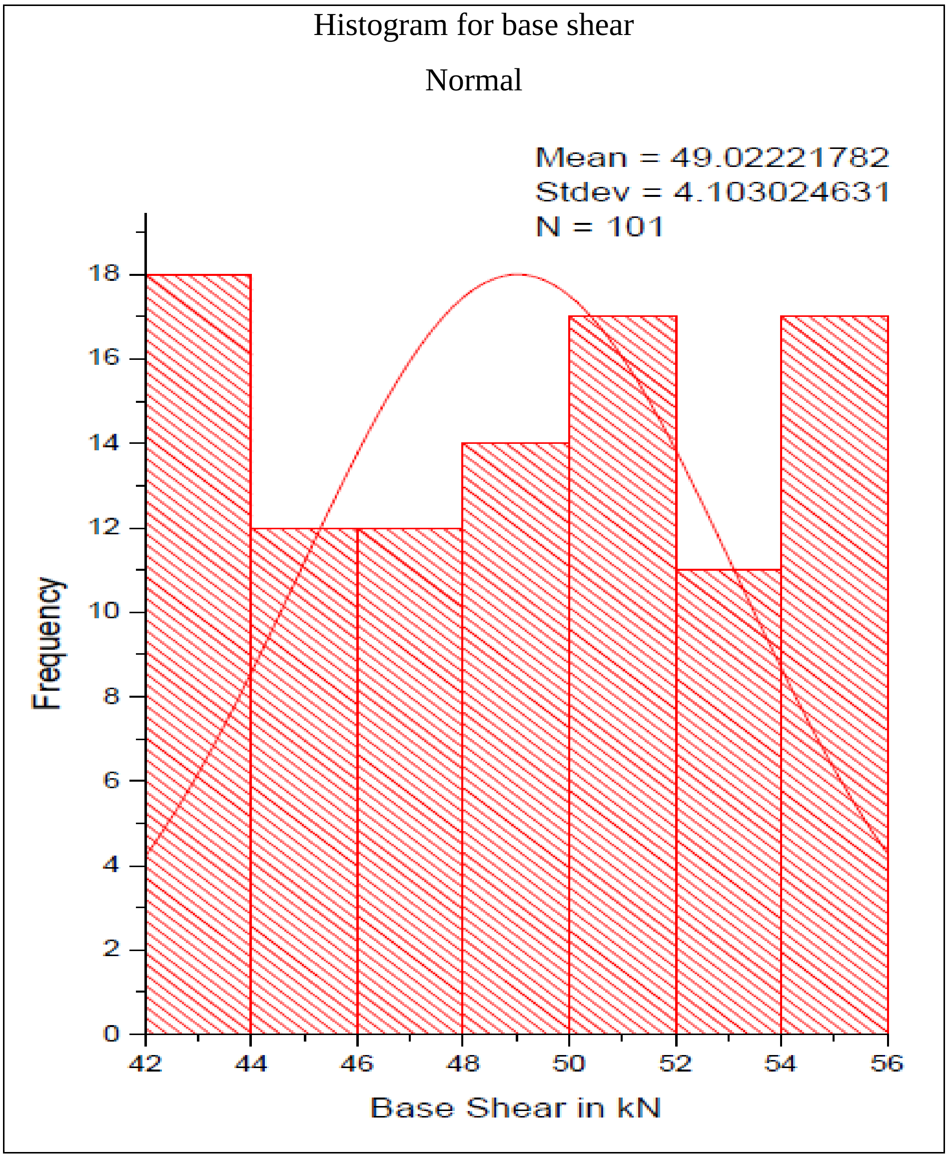


Fig. 4.13. Histogram for base shear generated for random values of steel strength fs, concrete strength fc and effective cover dc

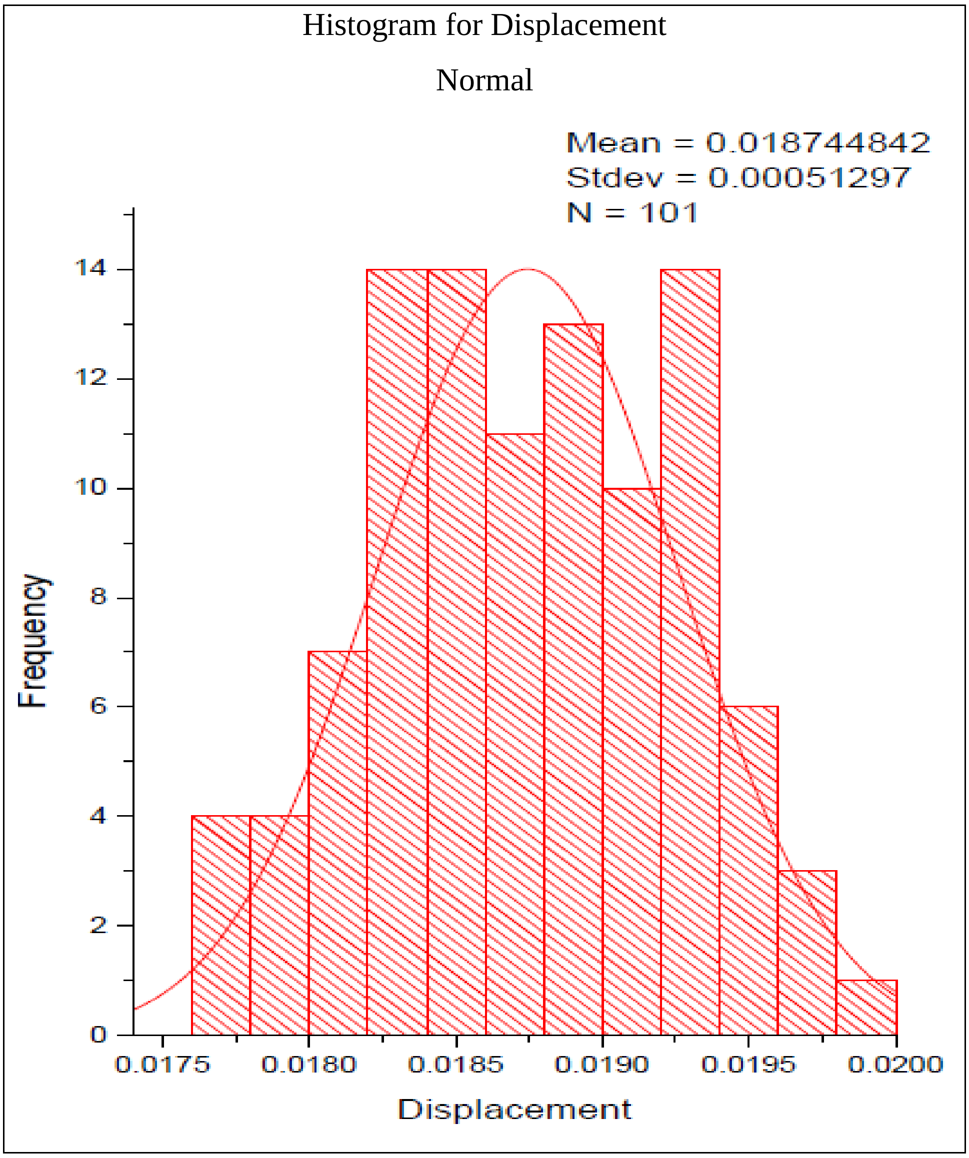


Fig. 4.14. Histogram for displacement generated for random values of steel strength fs, concrete strength fc and effective cover dc

4.4. INFLUENCE OF PLASTIC HINGE FORMATION SEQUENCE ON PUSHOVER ANALYSIS

In this section preselected potential hinge locations and random variations in strengths and covers have been considered to obtain plastic hinge formation sequences and bounds on displacements and base shears. Central values for concrete strength, steel strength and effective cover have been taken as 20MPa, 415MPa and 30mm. For this investigation beam and columns have been modeled as an assemblage of finite elements (12 each) to facilitate change in plastic hinge formation sequence by allowing variations (±15% from central values) in material strengths and disposition of reinforcements at potential plastic hinge locations. Analysis have also been performed by allowing random values for material strengths and effective covers within the specified range at possible potential plastic hinge locations without preselection.

4.4.1 Pushover analysis results for preselected potential plastic hinge locations and for randomly varying material strengths and effective covers

Analysis has been performed and results have been obtained for base shears and displacements considering material strengths and cover variation at preselected potential plastic hinge locations. Preselected potential plastic hinge locations were taken one at a time, two at a time, three at a time and a maximum possible of all four plastic hinge locations. When plastic hinge formation sequences started repeating, analysis has been terminated. Pushover analysis results have been presented in tables 4.9 - 4.12 giving details of maximum base shear and corresponding displacement and also plastic hinge formation sequence corresponding to plastic hinges with uncertainties associated

Table 4.9. Analysis results for randomly varying material strengths and effective covers at any one joint

Sl. No.	Hinges with uncertainties associated	Sequence Obtained	Base Shear 'kN'	Displacement 'm'
1	1	1342	32.561	0.014987
2	2	3421	39.388	0.015572
3	3	2413	28.887	0.017248
4	4	4231	30.283	0.016924
5	1	1423	31.694	0.018617
6	2	14 23	31.408	0.018799
7	3	41 23	31.671	0.017252
8	4	4123	31.966	0.018442
9	1	1432	31.931	0.018461
10	2	4132	31.170	0.018793
11	3	3214	39.290	0.015231
12	4	3142	39.562	0.015223

Table 4.10. Analysis results for randomly varying material strengths and effective covers at any two joints

Sl. No.	Hinges with uncertainties associated	Sequence Obtained	Base Shear 'kN'	Displacement 'm'
1	1,2	1243	35.217	0.013761
2	1,3	4123	43.119	0.022152
3	1,4	1324	30.231	0.015663
4	2,3	4321	35.214	0.017893
5	2,4	3214	31.689	0.039492
6	3,4	2431	39.774	0.044217
7	2,1	1423	32.391	0.018491
8	3,1	1432	31.607	0.018889
9	4,1	2314	31.266	0.018525
10	3,2	1423	33.145	0.021527
11	4,2	1232	32.126	0.019243
12	4,3	3241	39.268	0.021941

Table 4.11. Analysis results for randomly varying material strengths and effective covers at any three joints

Sl. No.	Hinges with uncertainties associated	Sequence Obtained	Base Shear 'kN'	Displacement 'm'
1	1,2,3	1432	46.821	0.048126
2	1,3,4	2134	33.431	0.014195
3	4,1,2	4213	41.245	0.043267
4	4,2,3	1234	30.567	0.014921
5	1,3,2	1423	42.972	0.045223
6	2,1,3	4132	39.170	0.018793
7	2,4,1	1423	31.408	0.0018713
8	4,2,1	4123	31.584	0.018799
9	3,1,4	3142	28.694	0.017186
10	3,2,1	2341	35.296	0.015645
11	3,4,2	1423	32.975	0.018720
12	2,1,4	3241	35.662	0.013219

Table 4.12. Analysis results for randomly varying material strengths and effective covers at all four joints

Sl. No.	Sequence Obtained	Base Shear 'kN'	Displacement 'm'
1	2314	35.976	0.0162782
2	1423	32.679	0.018420
3	4123	42.188	0.049421
4	2431	41.898	0.0501126
5	3214	41.299	0.0501264
6	4231	33.725	0.0192914
7	2314	40.196	0.0471267

Green and red colors indicate simultaneity in formation of plastic hinges

4.4.1.1 Results and discussions

The variation in base shear is 63% against variation in displacement of 264%, indicating the influence of plastic hinge formation sequence on analysis results and the need for its consideration in performance based designs. Such consideration allows analysts to establish bounds (as shown in table 4.13 and 4.14) on performances commensurate with uncertainties.

Table 4.13. Upper and lower bound of base shear values with associated displacements

Base Shear ‘kN’	Corresponding Displacement ‘m’	Sequence of Hinge formation
46.821	0.048126	1432
28.694	0.017186	3142

Table 4.14. Upper and lower bound of displacement values with associated base shears

Displacement ‘m’	Corresponding Base Shear ‘kN’	Sequence of Hinge formation
0.0501264	41.299	3214
0.013761	35.217	1243

4.4.2 Pushover results for random variations in material strengths and effective cover without preselection of number of possible potential plastic hinge locations

Analysis performed without preselection of number of potential plastic hinge locations to possess uncertainties (by allowing random values for material strengths and effective covers within the specified range) associated have yielded results as presented in table 4.15 and fig 4.8 gives pushover curves for all 24 sequences possible.

Table 4.15. Pushover analysis results for random variations in material strengths and effective covers at either one, two, three or all four possible potential plastic hinge locations

Sl. No.	**Sequence of hinge formation**	**Base Shear ‘kN’**	**Displacement ‘m’**
1	4132	48.985	0.050233
2	3412	45.893	0.048209
3	3142	42.781	0.051407
4	2413	42.879	0.046973
5	3124	42.65	0.043934
6	2431	42.872	0.046976
7	3214	36.923	0.04613
8	1432	48.992	0.050238
9	4213	45.285	0.047033
10	1423	45.314	0.047463
11	4123	47.437	0.019152
12	3421	44.856	0.019527
13	3241	44.59	0.020161
14	4312	49.179	0.019137
15	4231	35.83	0.013551
16	4321	36.672	0.015766
17	1243	36.402	0.015835
18	1324	35.462	0.013686
19	1342	35.529	0.013701
20	1234	30.426	0.015663
21	2143	39.154	0.015692
22	2134	39.458	0.01557
23	2314	36.921	0.015712
24	2341	37.106	0.015645

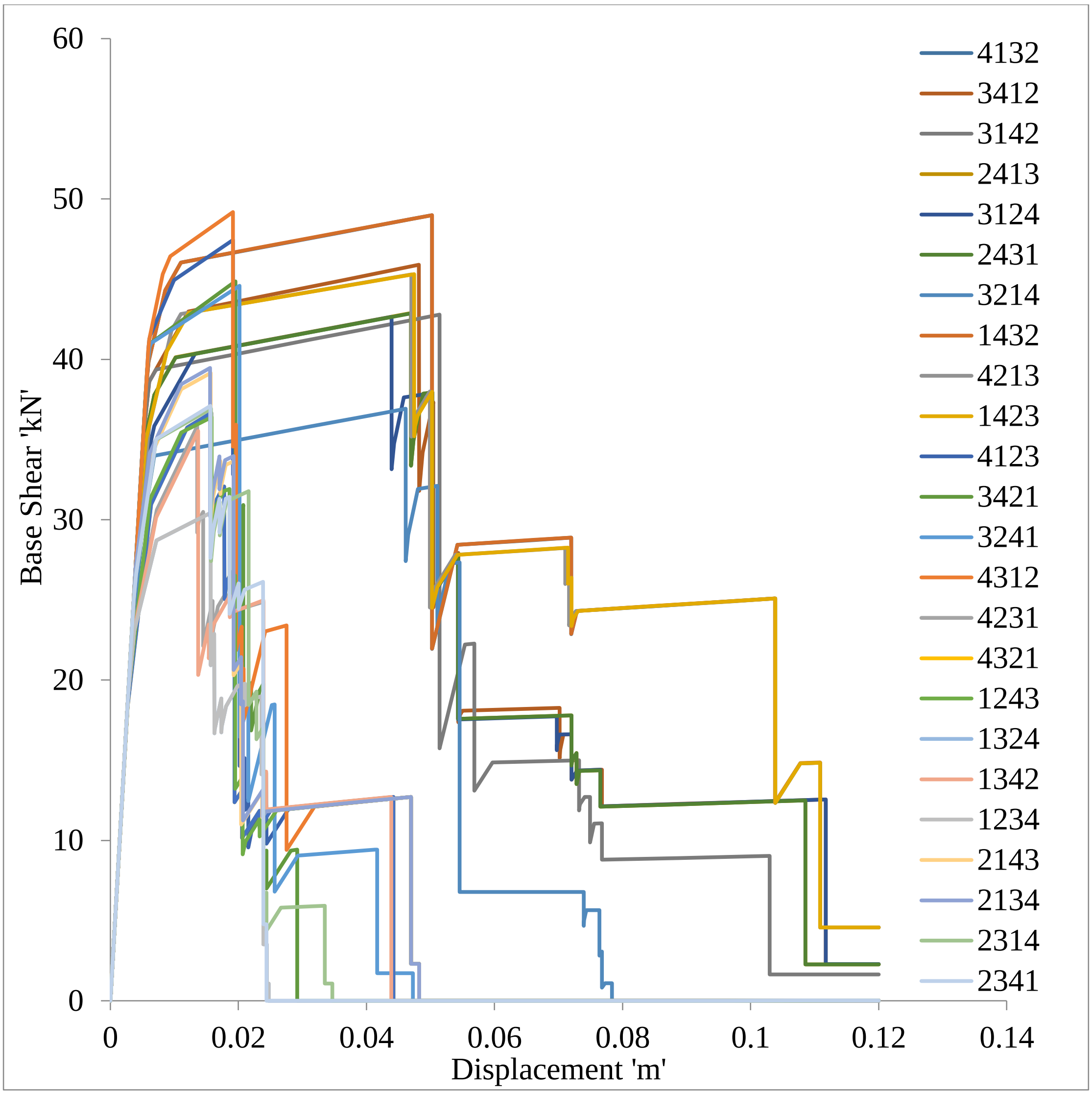


Fig. 4.15. Pushover curves for 24 sequences of plastic hinge formations

4.4.2.1 Results and discussions

From the investigation performed by allowing random values for material strengths and effective covers within the specified range without preselection of number of possible potential plastic hinge locations it can be observed that the minimum displacement obtained is 0.013551m and the corresponding base shear is 35.83kN for plastic hinge sequence 4231 and maximum displacement is 0.051407m with base shear of 42.781kN for plastic hinge sequence 3142.

For all other sequences, maximum base shear and corresponding displacement values are unique. This clearly brings out the importance of consideration of sequence of plastic hinge formation in pushover analysis to establish bounds on analysis instead of placing reliance on one analysis. It is also observed that the base shear variations are independent of changes in displacement characteristics.

Table 4.16. Statistical analysis results for randomly varying material strengths and effective covers at either one, two, three or all four possible plastic hinge locations

Output	Mean	Standard deviation	C.O.V '%'	95% Confidence Interval (CI)
Base Shear 'kN'	41.106	5.445	13%	(30.43 , 51.77)
Displacement 'm'	0.0295	0.0156	53%	(0.001 , 0.0601)

From table 4.16 the coefficient of variation for base shear and displacement being 13% and 53%, it is evident that displacement is more sensitive to plastic hinge formation sequence than base shear.

Considering material strengths and effective cover variations it has been shown in section 4.3.2.1 that displacement variation is 13%, whereas for the same frame plastic hinge formation sequence variation results in a change in displacement to the tune of 279%. Hence plastic hinge formation sequence is an important factor that should be considered and included in pushover analysis.

Inclusion of all possible plastic hinge formation sequence due to uncertainties in geometry, material strengths, and modeling techniques calls for tremendous computational time and efforts. Hence strategies have to be proposed, formulated and tested. An attempt in this direction made is elaborated in chapter 5.

CHAPTER 5

STRATEGIES TO ADDRESS UNCERTAINTIES IN PUSHOVER ANALYSIS RESULTS DUE TO PLASTIC HINGE FORMATION SEQUENCE

5.1 CHAPTER PROLOGUE

The influence of plastic hinge formation sequence on analysis results has been highlighted in the preceding sections. Its consideration in performance appraisal hence is very important.

Inclusion of all possible plastic hinge formation sequence due to uncertainties in geometry, material strengths, and modeling techniques calls for tremendous computational time and efforts.

Strategies have been suggested, formulated adopted and validated by way of comparison of results of the analytical investigations with experimental test results.

5.2 DETAILS OF THE EXPERIMENTAL PUSHOVER TEST DATA AVAILABLE

A three storied, RC frame model structure as in fig 5.1(a) and 5.1(b), has been tested at Structural Engineering Research Center, Chennai. The structure consisted of three stories and two bays along two orthogonal directions. The beam and column sections of the model are as detailed in fig 5.2. The height of the floor is 1800mm for all storeys and bay widths are 1500mm each. The slab is 50mm thick. The average concrete strength and average reinforcement yield strength of the tested structure are 35MPa and 478MPa. The pushover test has been performed under monotonically increasing lateral pushover loads till failure. The structure has been gradually pushed by small increments of loads and corresponding displacements have been recorded. The lateral load distribution across various levels has been arrived at based on FEMA 356 specifications.

Fig. 5.1(a). Experimental setup for pushover analysis

Fig. 5.1(b). Photograph of actual tested structure (photograph reprinted from Thapa M, 2009)

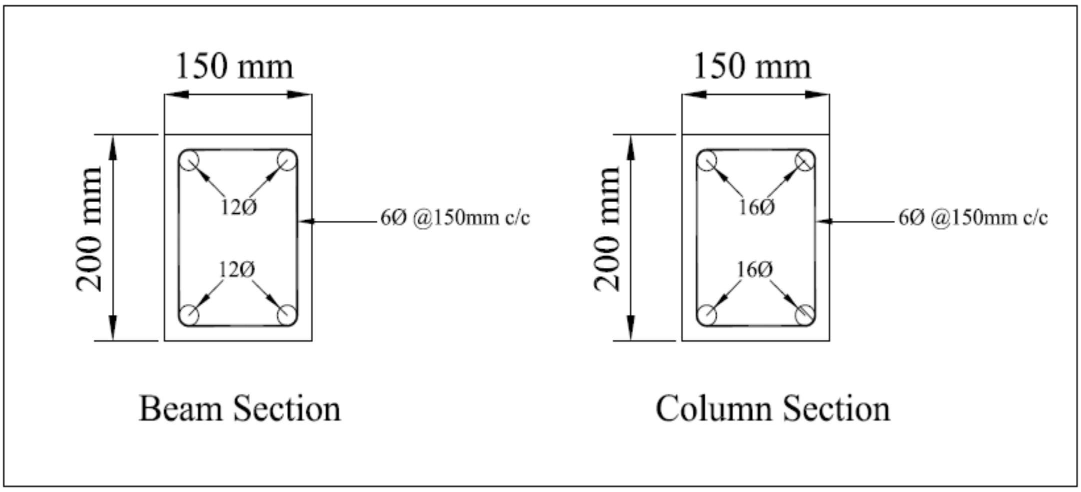


Fig. 5.2. Section properties of actual tested structure (Thapa. M., 2009)

5.2.1 Load pattern

Lateral load distribution across the height of the building has been found out as shown below,

Measured values;

Height of the first floor from foundation level, $h1 = 1800$mm

Height of the second floor from foundation level, $h2 = 3600$mm

Height of the third floor from foundation level, $h3 = 5400$mm

Calculated values;

$\sum hi^2 = 1800^2 + 3600^2 + 5400^2 = 45360000$

Weightage of total force for the first floor $= h1^2 / \sum hi^2 = 1800^2 / 45360000 = 0.0714$

Weightage of total force for the second floor $= h2^2 / \sum hi^2 = 3600^2 / 45360000 = 0.2857$

Weightage of total force for the third floor $= h3^2 / \sum hi^2 = 5400^2 / 45360000 = 0.6429$

During the test, the load has been increased monotonically in the ratio 0.6429:0.2857:0.0714 ≈ 9: 4: 1 for the roof: second floor: first floor as shown in fig 5.3.

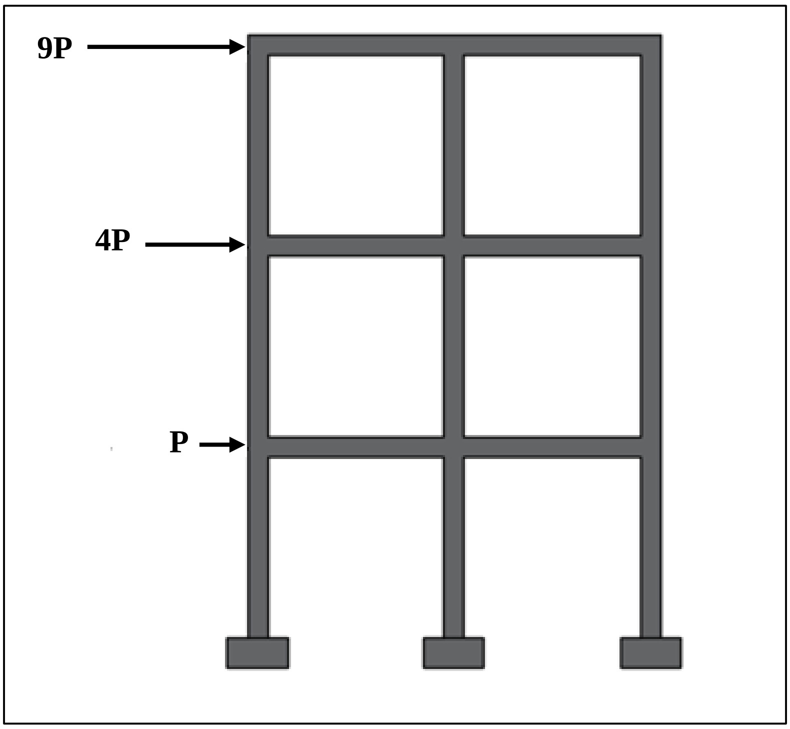


Fig. 5.3. Details of loading pattern obtained from experimental study (Thapa. M., 2009)

Pushover curve obtained from the test is as shown in fig 5.4. The maximum base shear and corresponding displacement are 286.5kN, 0.11m respectively.

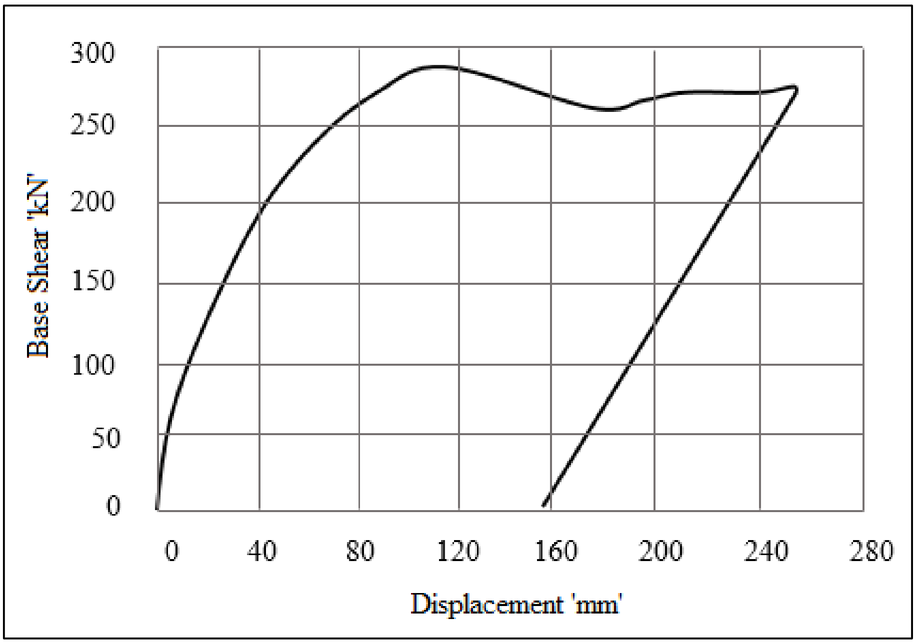


Fig. 5.4.Pushover curve obtained from experimental study (Sharma. A., 2008)

5.3 ANALYTICAL INVESTIGATION DETAILS - APPROACHES TO STRATEGIZE PLASTIC HINGE FORMATION SEQUENCE STUDIES

The structure in all has 36 beam column junctions which have been considered as potential plastic hinge locations. The number of possible plastic hinge sequences is 36! (3.7199×10^{41}). It is impossible and impractical to consider all possibilities.

And hence formulation of strategies are necessary to reduce computational time and efforts. The following sections give details of strategies proposed, formulated, adopted and validated.

Efficacy of strategies proposed have been verified by comparing analysis results with experimental pushover test data available for a model frame.

5.3.1 Strategy Formulation

1. It has been proposed to limit the uncertainties in material strengths and discrepancies in positioning of reinforcement to ± 15% of specified.
2. Number of plastic hinge locations with uncertainties associated, have been restricted to 15% of total number of potential plastic hinge locations considered at beam column joints.
3. In the first phase of strategization, plastic hinge locations with uncertainties have been preselected at potential plastic hinge locations in horizontal planes at floor levels and in vertical planes at beam column joints, restricting plastic hinge locations with uncertainties to two dimensions i.e. either in specific horizontal and vertical planes.
4. In the next phase, plastic hinge locations with uncertainties associated have been randomly distributed in the 3D frame.
5. Pushover analysis has been performed for aforesaid cases. Plastic hinge formation sequences have been monitored and the results have been presented and interpreted.

5.4 PUSHOVER ANALYSIS OF TESTED STRUCTURE WITHOUT ALLOWANCE FOR UNCERTAINTIES (S0)

Pushover analysis has been carried out on the structure model for which experimental results are available. For all the pushover analysis, SAP2000 version 14 has been used. The moment curvature relationship for moment M3 hinges considered is as presented in chapter 4, fig 4.2.

Fig 5.5 shows the pushover curve for the structure considering geometry, material strength and element section details as provided in the experimental test data. No uncertainties have been considered. The maximum base shear and corresponding displacement are 283.6kN, 0.099m respectively.

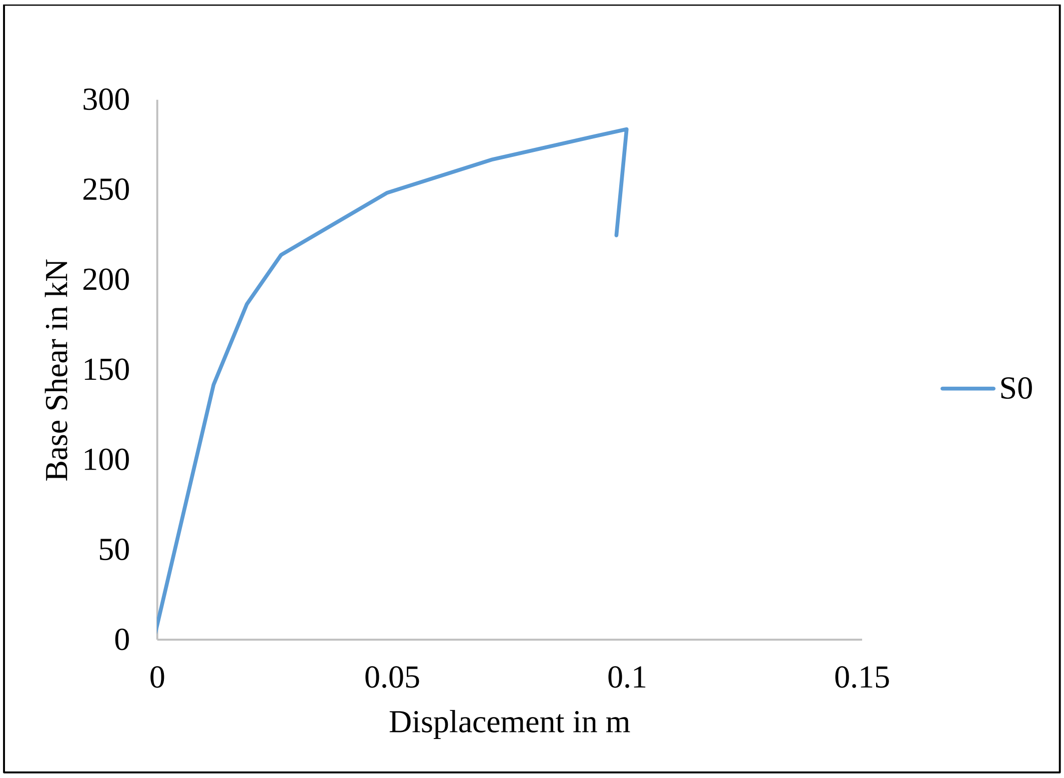


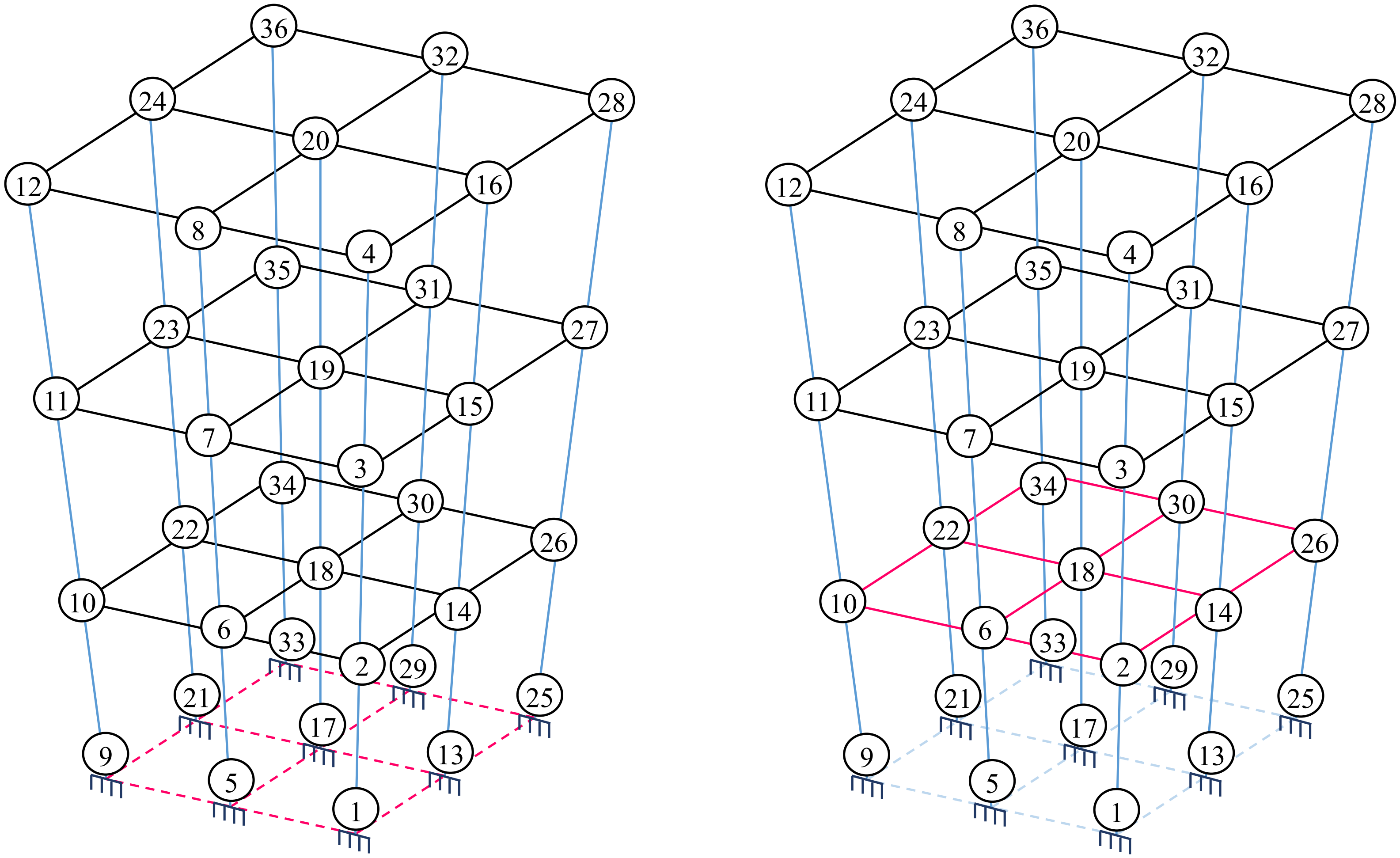
Fig. 5.5. Pushover curve for analytical model structure without allowance for uncertainties

5.5 RESULTS OF PUSHOVER ANALYSIS ADOPTING STRATEGY 1 (S1) AND STRATEGY 2 (S2)

As strategy 1 and strategy 2, pushover analysis has been performed considering 6 preselected locations (say about 15% of total number of potential plastic hinge locations) to have uncertainties associated with material strengths and effective cover to steel reinforcement placement. Preselected locations are either in specific horizontal or vertical planes.

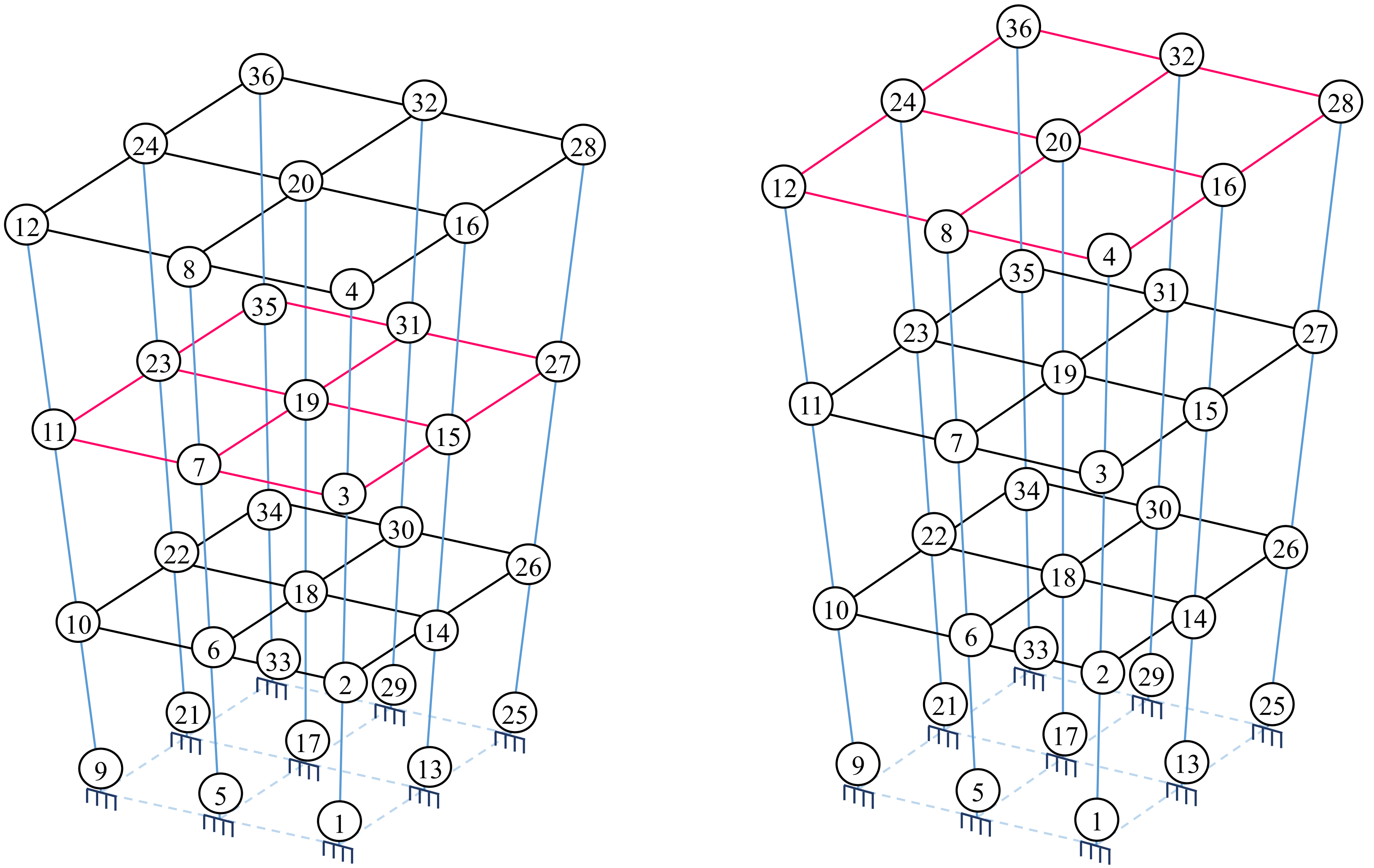
5.5.1 Strategy 1 – Uncertainties restricted to horizontal planes (S1H)

Fig 5.6 shows locations of plastic hinges with uncertainties associated, all considered in one horizontal plane per analysis and have been coded as S1H0, S1H1, S1H2, and S1H3. The pushover analysis results along with sequence of plastic hinge formations have been presented in table 5.1 and the same has been shown in fig 5.7. Pushover curves obtained for S1H analysis are as shown in fig 5.8.



(a) S1H0 **(b) S1H1**

Fig. 5.6. Location of plastic hinges in specified horizontal planes



(c) S1H2 **(d) S1H3**

Fig. 5.6. Location of plastic hinges in specified horizontal planes

Table 5.1. Pushover analysis results obtained by employing strategy 1 (S1H)

Sl. No.	Analysis Code	Base shear 'kN'	Displacement 'm'	Sequence of Plastic hinge formations (Circle with fill indicates plastic hinges with uncertainities associated)				
				Step 1	Step 2	Step 3	Step 4	Step 5
1	S1H0	281.7	0.0993	2,10,34, 26	6,22,30,14, 18,3,7,11,15, 19,23,27,31, 36	1 13 25 17 5, 29,21	33 9 4,16,28,20, 12,24,35	8,32
2	S1H1	290.6	0.1006	6	14 22 2,18,34,3,7, 11,23,35,31, 27,15,19	26 10 30 9,21,33,29, 25,13,17	1,5,12,24,36 ,8,20,4,16, 28	32
3	S1H2	283.1	0.1004	3	7 11 35 31,27,2,6,10, 22,34,30,26, 14	15 19 23,18,1,5,9, 13,17,21,25, 29,33	4,8,12,16,20 ,24,28,32,36	-
4	S1H3	283.6	0.0992	2,10, 34,26	6,22,30,18, 14,3,7,11,23, 35,31,27,15, 19	1,5,9,13,17, 21, 25,29,33	4 20 24 28 12,36, 16,	8 32

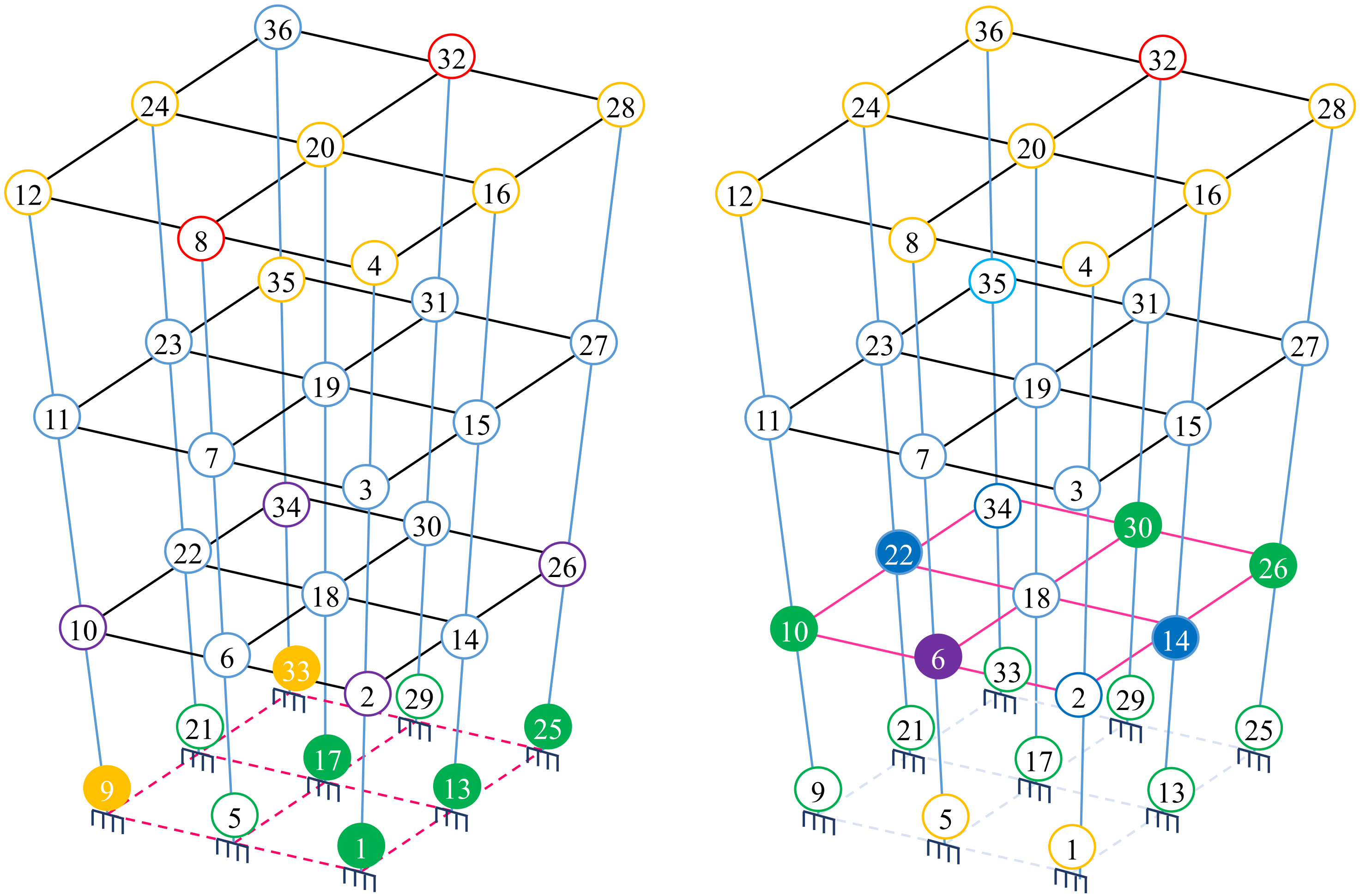
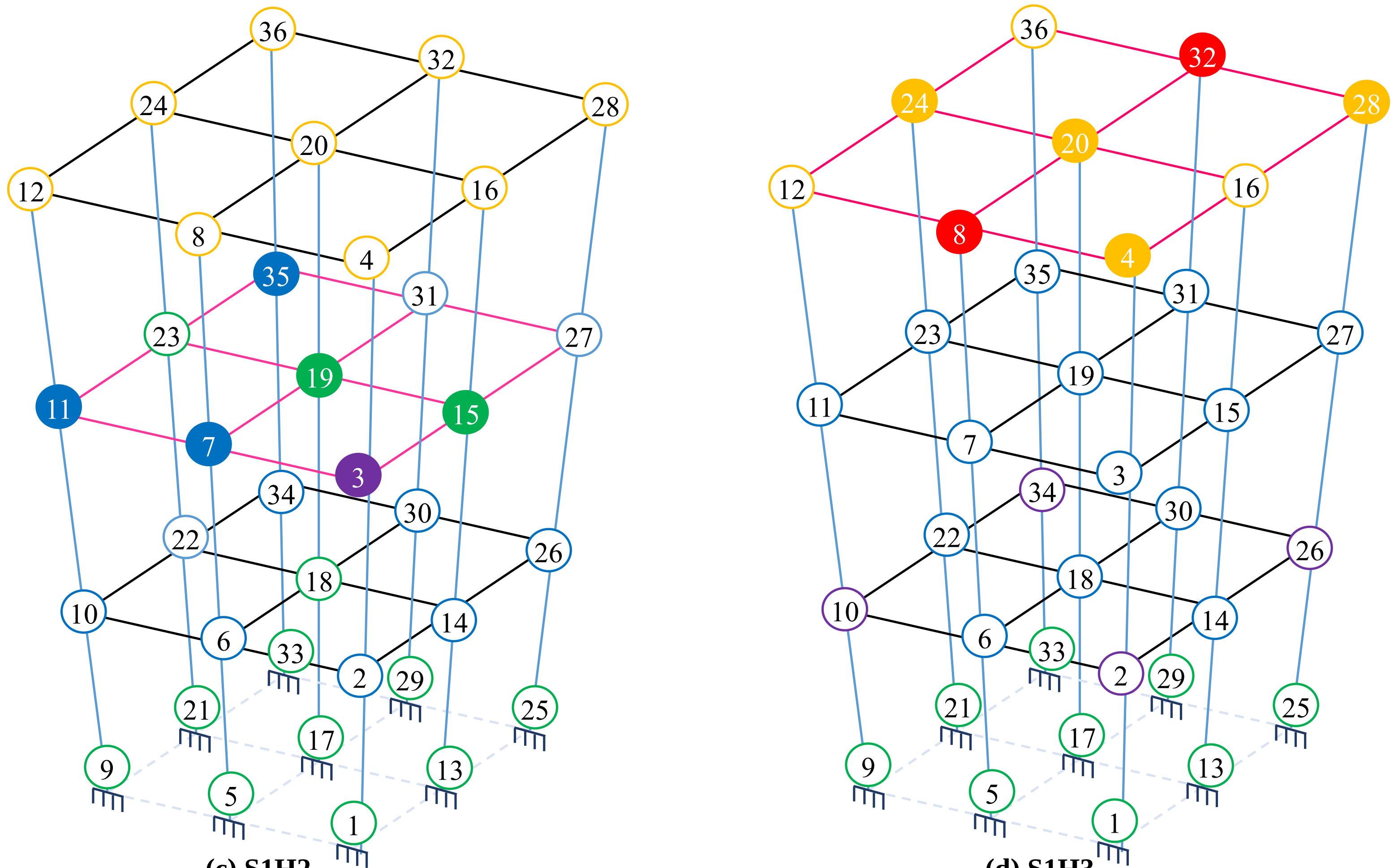


Fig. 5.7. Sequence of plastic hinge formations with associated uncertainties at preselected locations in specified horizontal planes

○Plastic hinges formed in step 1 ○Plastic hinges formed in step 2 ○Plastic hinges formed in step 3 ○Plastic hinges formed in step 4 ○Plastic hinges formed in step 5
and Circles with fill indicate plastic hinges with associated uncertainties associated



(c) S1H2 (d) S1H3

Fig. 5.7. Sequence of plastic hinge formations with associated uncertainties at preselected locations in specified horizontal planes

○Plastic hinges formed in step 1 ○Plastic hinges formed in step 2 ○Plastic hinges formed in step 3 ○Plastic hinges formed in step 4 ○Plastic hinges formed in step 5

and Circles with fill indicate plastic hinges with uncertainties associated

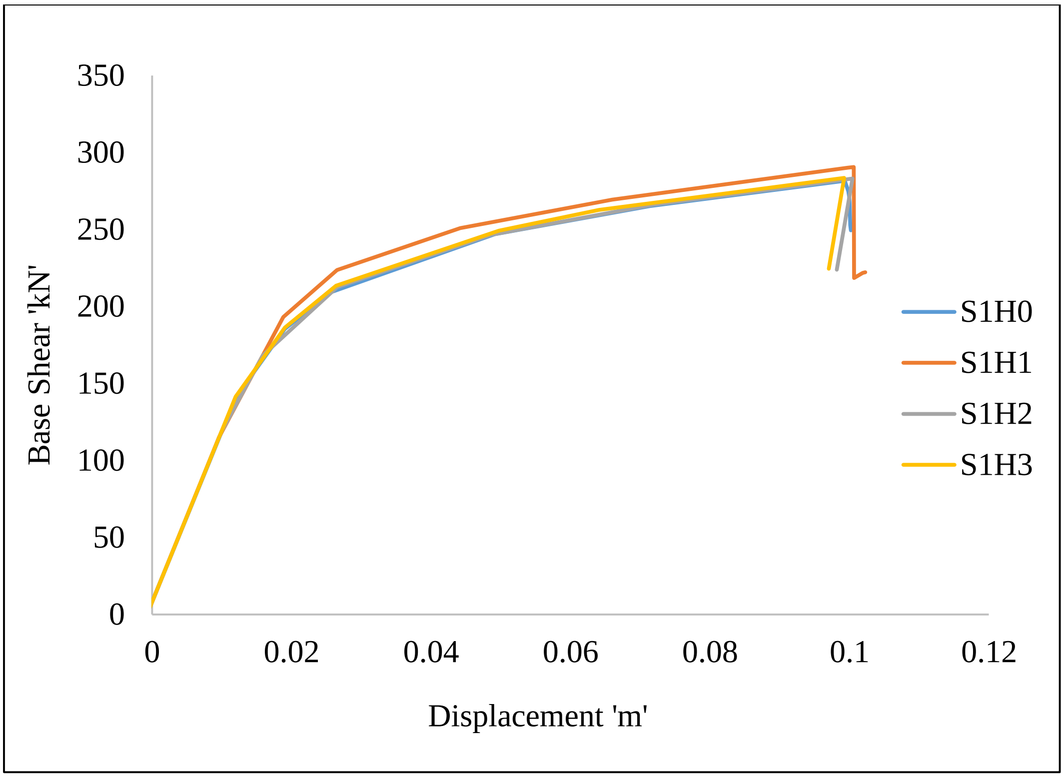
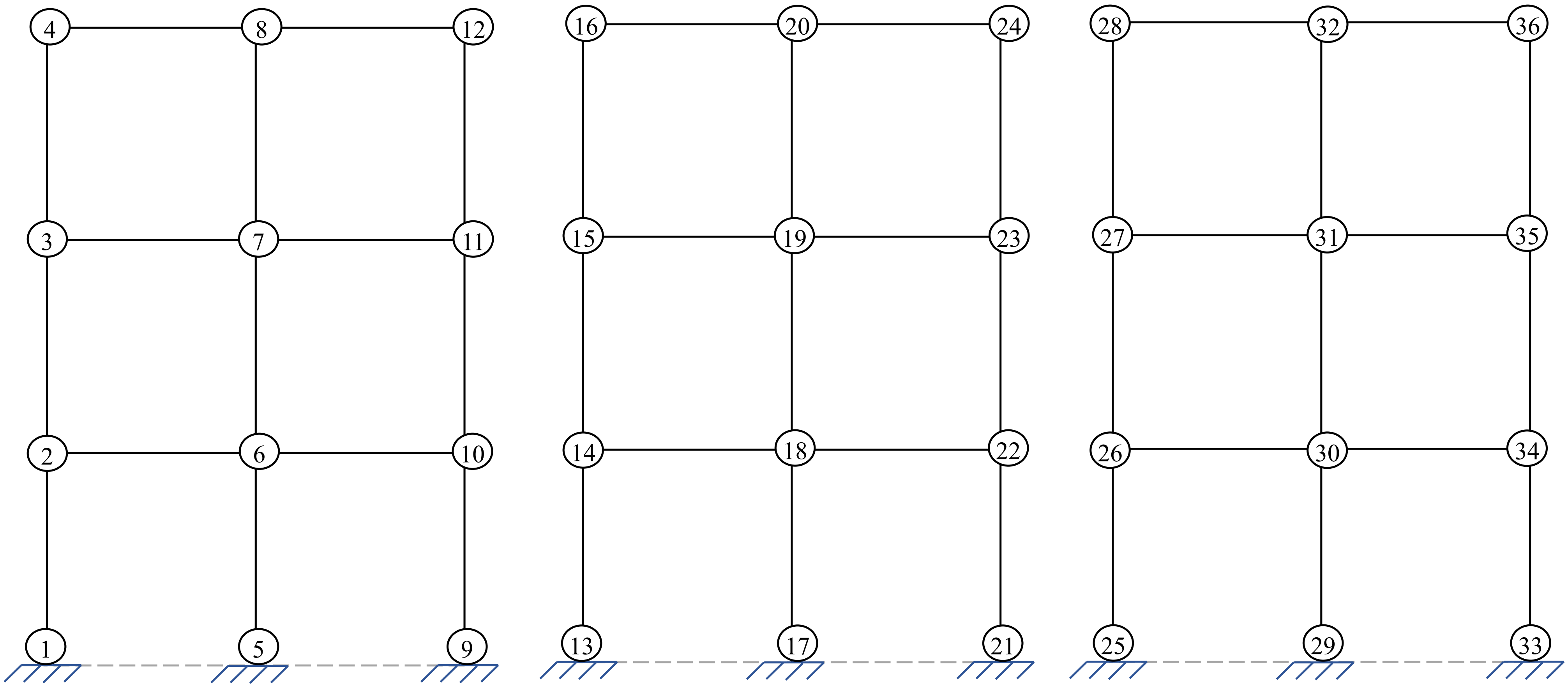


Fig. 5.8. Pushover curves obtained for Strategy 1(S1H)

5.5.2 Strategy 1 - Uncertainties restricted to vertical planes (S1V)

Locations of plastic hinges with uncertainties associated, have been considered in one vertical plane per analysis and have been coded as S1V1, S1V2, and S1V3 as in fig 5.9. The pushover analysis results and sequence of plastic hinge formations are tabulated in table 5.2 and the same has been shown in fig 5.10. Pushover curves obtained for S1V analysis is shown in fig 5.11.

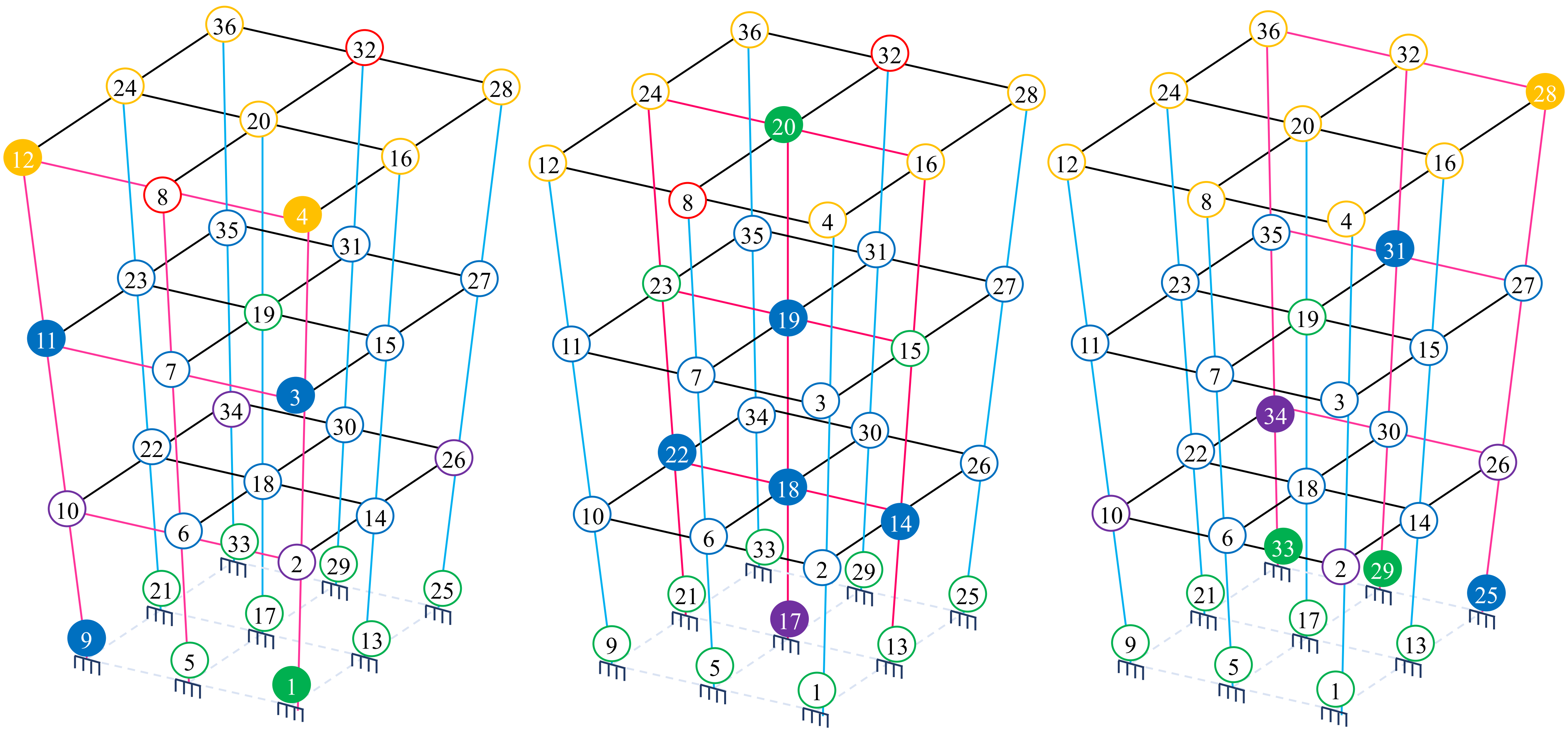


(a) S1V1 **(b) S1V2** **(c) S1V3**

Fig. 5.9. Location of plastic hinges in specified vertical planes

Table 5.2. Pushover analysis results obtained by employing strategy 1 (S1V)

Sl. No	Analysis Code	Base shear ‘kN’	Displacement ‘m’	Sequence of Plastic hinge formations (Circle with fill indicates plastic hinges with uncertainities associated)				
				Step 1	Step 2	Step 3	Step 4	Step 5
1	S1V1	276.5	0.0933	2,10,34, 26	3 9 11 22,18,14,30, 6,7,23,35,31 ,27,15	1 19,2,13,17, 21,25,29,33	4 12 24,36,20,16, 28	8,32
2	S1V2	266.5	0.0958	17	14 18 19 22 37,31,2,6,10 ,34,26,30,3, 7,11,35	20 15,23,1,5,9, 13,21,25,29, 33	4,16,28,36,2 4,12	8,32
3	S1V3	278.4	0.0944	34 2,10,26	25 31 6,22,30,14, 18,3,7,11,23 ,35, 27,15	29 33 19,13,1,5,9, 21,17	28 4,8,12,16,20, 24,32,36	-



(a) S1V1 **(b) S1V2** **(c) S1V3**

Fig. 5.10. Sequence of plastic hinge formations with associated uncertainties at preselected locations in specified vertical planes

○Plastic hinges formed in step 1 ○Plastic hinges formed in step 2 ○Plastic hinges formed in step 3 ○Plastic hinges formed in step 4 ○Plastic hinges formed in step 5
and Circles with fill indicate plastic hinges with uncertainties associated

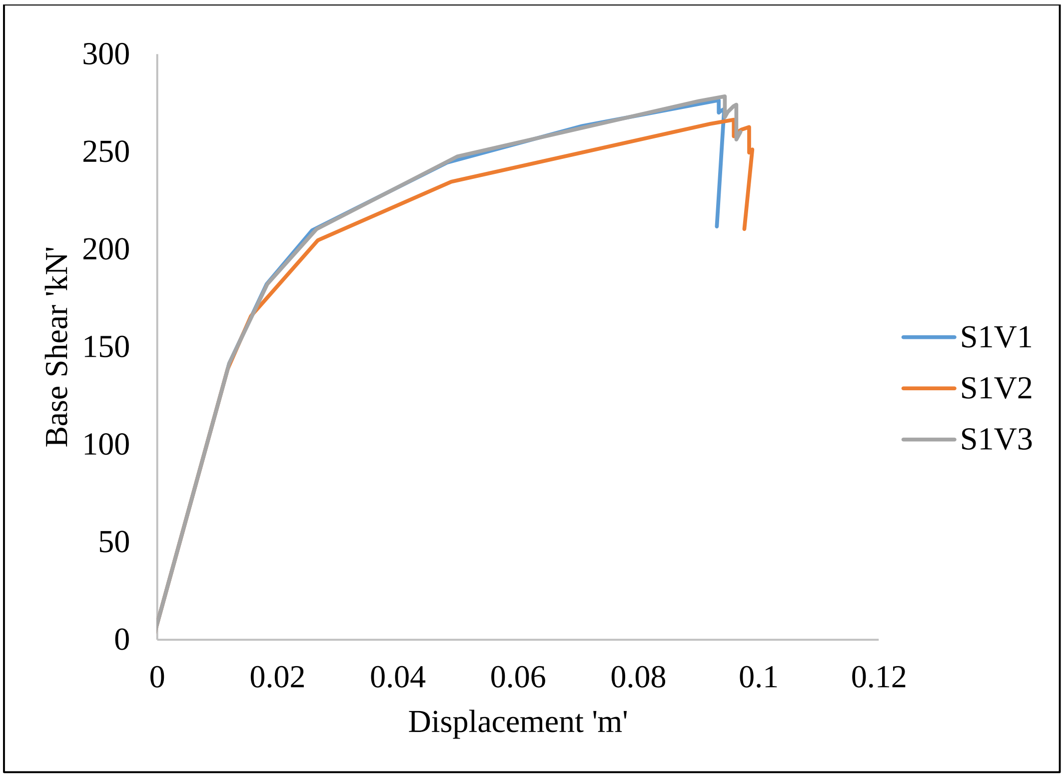


Fig. 5.11. Pushover curves obtained for Strategy 1(S1V)

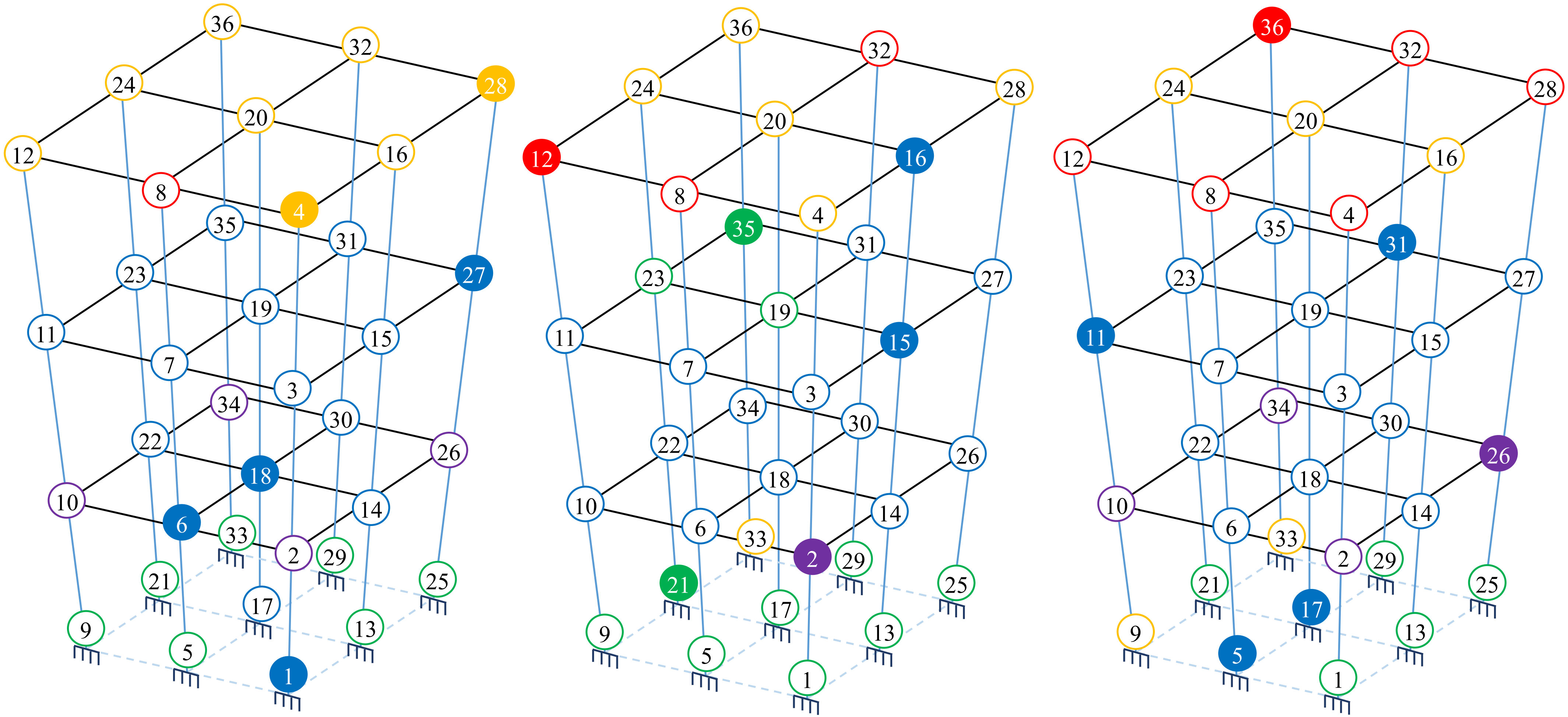
Base shears and displacement obtained are greater for S1H than S1V with results for S1H series closer to experimental.Separation of pushover curve in the nonlinear range is more pronounced for S1V series

5.5.3 Results of pushover analysis adopting Strategy 2 (S2R)

Strategy 2 adopts randomization of plastic hinge locations with uncertainties associated, distributed throughout the 3D frame. Pushover analysis results and sequence of plastic hinge formations obtained for three random generations coded as S2R1, S2R2 and S2R3 are detailed in table 5.3 and shown in fig 5.12. Pushover curves obtained for S2R analysis are as shown in fig 5.13.

Table 5.3. Pushover analysis results obtained by employing strategy 2 (S2R)

Sl. No	Analysis Code	Base shear 'kN'	Displacement 'm'	Sequence of Plastic hinge formations (Circle with fill indicates plastic hinges with uncertainities associated)				
				Step 1	Step 2	Step 3	Step 4	Step 5
1	S2R1	284.7	0.1094	2,10,26, 34	1 6 18 27 22,30,14, 17,3,7,11,15 ,19,23,31,36	5,9,21,13 ,25,29,33	4 28 16,20,32, 12,24,36	8
2	S2R2	283.2	0.1024	2	15 16 31,14,26,6,1 0,18,22,30,3 4,3,7,11, 27	21 35 23,19, 1,5,9,13, 17, 25,29	33,4, 28,20, 24,36	12 8,32
3	S2R3	279.8	0.0956	26 2,10,34,	5 11 17 31 7,19,14,6, 22,30,18,3, 15,27,35,23	1,13,25, 29,21	9,33, 16,20, 24	36 28,32, 4,8, 12



(a) S2R1 (b) S2R2 (c) S2R3

Fig. 5.12. Sequence of plastic hinge formations with associated uncertainties at random locations throughout the 3D frame

○Plastic hinges formed in step 1 ○Plastic hinges formed in step 2 ○Plastic hinges formed in step 3 ○Plastic hinges formed in step 4 ○Plastic hinges formed in step 5

and Circles with fill indicate plastic hinges with uncertainties associated

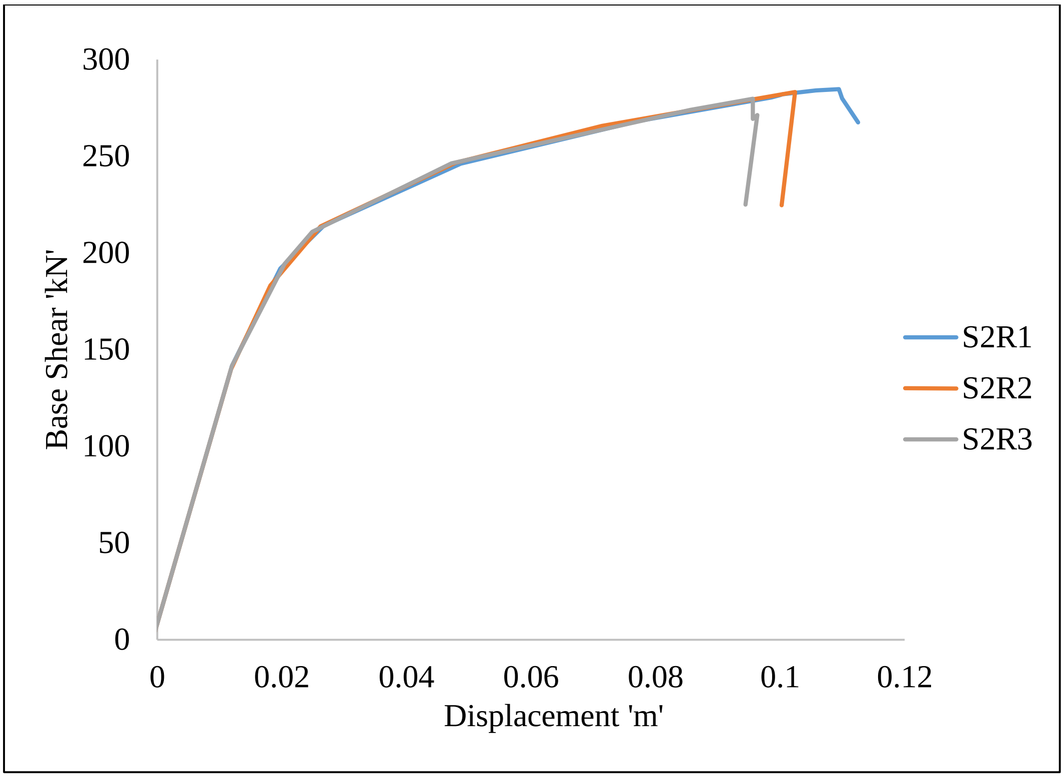


Fig. 5.13. Pushover curves obtained for strategy 2 (S2R)

The analysis results for maximum base shear and corresponding displacement almost perfectly match experimental values with discrepancies less than a percent.

5.6 APPRAISAL OF SUGGESTED STRATEGIES

Analysis assuming specified material strengths and detailing without uncertainties has yielded 1% lower base shear and 10% lower displacement values in comparison to experimental results.

Strategy 1 indicates when hinges with uncertainties are restricted to horizontal planes, base shear value are lower by 1.7% and higher by about 1.4% whereas displacements are lower by about 10%. When hinges with uncertainties are restricted to vertical planes, the difference in both base shear and displacements are to the tune of 7% and 15% on the lower side respectively, suggesting S1H strategy is superior to S1V.

Table 5.4. Comparisons of analysis results with experimental results

Sl. No.	Analysis Code	Base Shear		Displacements	
		Lowest %	Highest %	Lowest %	Highest %
1	S0	-1.012	-	-9.99	-
2	S1H	-1.68	+1.43	-9.818	-8.5454
3	S1V	-6.98	-2.827	-15.181	-13.909
4	S2R	-2.33	-0.62	-13.09	-0.545

From table 5.4 it can be seen that, strategy 2 wherein hinges associated with uncertainties are randomized in the 3D frame, provides best estimates for both maximum base shear and corresponding displacement values that are closest to experimental results.

5.7 SECANT STIFFNESS EVALUATION AND STRATEGY EFFICACY COMPARISON

Best estimates of secant stiffness's computed for maximum base shear and corresponding displacements have been compared with experimental value. Results are as presented in table 5.5. Values presented are for the cases considering no uncertainty and uncertainties restrained to horizontal and vertical planes and also for randomly distributed locations throughout the 3D frame. Fig 5.14 shows pushover curves obtained for experimental study and also for analytical investigations that are closest to experimental result.

It can clearly be seen that slope of the initial part of the curve is steeper for all the simulated models in comparison to the slope of experimental curve. This may be attributed to the test frame being more flexible than the simulated models owing to partial fixity at base, material deficiencies and defects in workmanship.

Table 5.5. Secant stiffness results from analysis in close agreement with experimental data

Sl. No.	Secant Stiffness Code (Keff)	Secant Stiffness Value kN/m
1	Keff-Experimental	2604.54
2	Keff-S0	2864.64
3	Keff-S1H2	2819.72
4	Keff-S1V2	2781.83
5	Keff-S2R1	2602.37

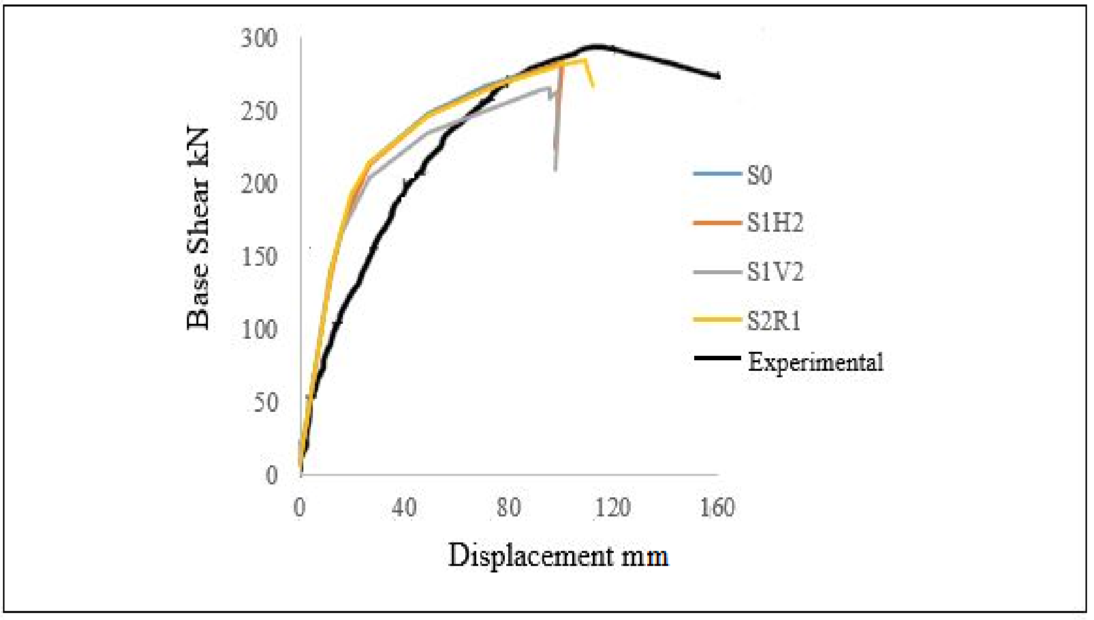


Fig. 5.14. Pushover curves obtained for experimental study and for analytical investigations that are closest to experimental result

From the secant stiffness values, obtained and reported in table 5.5 it can be concluded that consideration of plastic hinge formation sequence in pushover analysis allowing for uncertainties in material strengths and effective cover to steel helps in prediction of behavior closer to reality.

5.8 SUMMARY

To reduce computational efforts associated with consideration of the influence of plastic hinge formation sequence on pushover analysis results, strategy 1 and 2 have been formulated and attempted.

Results obtained from strategy 2, wherein randomization of plastic hinge locations with uncertainties associated, distributed throughout the 3D frame are in very close agreement with experimental values indicating its efficacy and utility.

CHAPTER 6

CONCLUSIONS

The influence of sequence of formation of plastic hinges on pushover analysis has been investigated by considering uncertainties and the following have been accomplished.

- Importance of consideration of plastic hinge formation sequence has been illustrated by simple example of a propped cantilever beam loaded at center with a concentrated load.

  The invariance of the collapse load to plastic hinge formation sequence and the drastic change in load displacement characteristics have been highlighted to justify the need for consideration of sequence of plastic hinge formation in pushover analysis.
- Plastic hinge formation sequence can alter due to uncertainties associated with material strengths and geometry. To understand the effect of these parameters, analyses have been performed on single bay single storey RC frame. Material strengths variations and cover to reinforcement have been allowed to vary plus or minus 15% of design values. Discrete values for material strengths and covers within this specified range have been adopted and also random values have been considered for investigation.

  From the results of analyses it is clear that base shear and displacement values are very sensitive to change in strength of steel and not to concrete strengths and cover. This is due to fact that the RC sections of elements are all under reinforced.

  Pushover analysis results obtained by permitting 15% allowable range of material strengths variations randomly at preselected locations, yielded in an increase in displacement of 13% over that for design values. Whereas by randomizing both uncertainties and number of possible potential plastic hinge locations to have these uncertainties, displacement changed by 279% indicating sequence of plastic hinge formation is the most influential factor which affects the displacement characteristics in pushover analysis. Bounds on displacements for possible plastic hinge formation sequences hence can be obtained from analysis of this kind which help immensely in performance appraisal.

- Having established the importance of plastic hinge formation sequence's influence on pushover analysis results by examples of propped cantilever beam and single bay single storey RC frame, and its utility in performance appraisal, its consideration has been extended to multistory multibay frame.

  As the number of probable plastic hinge locations increases, number of possible sequences becomes mind boggling. Investigation of all possible sequences due to uncertainties associated with material properties and workmanship calls for tremendous computational time and efforts.

  Strategies have been proposed formulated employed and their efficacy has been demonstrated for establishment of bounds on analysis results by comparing with available results of experimental pushover test.
- Strategy 1 considers 15% of potential plastic hinge locations and allows variations leading to different sequences. Here again two sub strategies have been proposed by way of allocation of defective hinge locations to horizontal and vertical planes.
- When hinges with uncertainties are restricted to horizontal planes, base shear value are lower by 1.7% and higher by about 1.4% whereas displacements are lower by about 10%. When hinges with uncertainties are restricted to vertical planes, the difference in both base shear and displacements are to the tune of 7% and 15% on the lower side respectively, suggesting S1H strategy is superior to S1V
- Strategy 2 adopts randomization of plastic hinge locations with uncertainties associated, distributed throughout the 3D frame. The analysis results provides the best estimate as compared to strategy 1 for behavior with both base shear and displacement values almost perfectly matching experimental values with discrepancies less than a percent.
- Strategy 1, finds application where defect and deficiency features are known a priori. Whereas strategy 2 can be employed in all situations.

- Availability of experimental data on pushover tests where plastic hinge formation sequence has also been monitored, recorded and available shall help in fine tuning the approaches suggested.

REFERENCES

Abbie, B. Liel., Curt, B. Haselton., Gregory, G. Deierlein and Jack, W. Baker (2009). "Incorporating modelling uncertainities in the assessement of seismic collapse risk of buildings." *Structural Safety.,* 31, 197-211.

Ahmed Sofyan. Y, (2013). "Seismic Evaluation of Reinforced Concrete Frames Using Pushover Analysis." *Al-Rafidain Engineering.,*21(3), 28-45.

Alessandra Fiore., Girolamo Spagnoletti and Rita Greco. (2016). "On the prediction of shear brittle collapse mechanisms due to the infill-frame interaction in RC buildings under pushover analysis." *Engineering Structures.,* 121, 147–159.

ATC, "NEHRP Guidelines for the Seismic Rehabilitation of Buildings, FEMA 273 Report." by the Applied Technology Council for the Building Seismic Safety Council, published by the Federal Emergency Management Agency, Washington, D.C. 1997a.

ATC-24 (Applied Technology Council). (1992). "Guidelines for cyclic seismic testing of components of steel structures", California.

ATC-40 (Applied Technology Council). (1996) "Seismic evaluation and retrofit of concrete buildings", California.

Bagchi, A. (2004). "A Simplified method for evaluating the seismic performance of buildings." *Earthquake Engineering and engineering vibrations*., 3(02),223-236.

Balogh T and Vigh L.G, (2012). "Genetic algorithm-based optimization of regular steel building structures subjected to seismic effects." *15 WCEE., lisbo*.

Bidoki Reza Zare and Mohsenali Shayanfar. (2018). "An energy-based pushover-analysis with torque-effects in assessment of the structures with asymmetric plan." *Soil Dynamics and Earthquake Engineering.,* 108,58–68.

Borros, R.C. and Almeida, R. (2005). "Pushover analysis of asymmetric three-dimensional building frame." *Journal of civil engineering and management.,* 9, 3-12.

Chopra A.K and Goel R.K. (2004). "A modal pushover analysis procedure to estimate seismic demands for unsymmetric-plan buildings." *Earthquake engineering and structural dynamics.,* 33, 903–927.

Chopra A.K and Goel, R.K. (2001). "A Modal Pushover Analysis Procedure to Estimate Seismic Demands for Buildings: Theory and Preliminary Evaluation." Pacific Earthquake Engineering Research Center, College of Engineering University of Berkeley.

Chopra AK. (2007). "Dynamics of structures: theory and applications to earthquake engineering." Prentice-Hall international series in civil engineering and engineering mechanics.

CSI, SAP2000. (2002). "Integrated finite element analysis and design of structures basic analysis reference manual." Berkeley (CA, USA): Computers and Structures.

D'Ambrisi A, Mezzi M. (2015). "An energy-based approach for nonlinear static analysis of structures." *Bull Earthq Eng.*,13(5):1513–30.

Deierlein G., Hsieh S.H., (1990). "Seismic response of steel frames with semi-rigid connections using the capacity spectrum method." *Proceedings 4th US National Conference on Earthquake Engineering.*, 2, 863-72.

Fajfar P.A. (2000). "A non-linear analysis method for performance based seismic design." *Journal of Earthquake Spectra*.,16, 573-592.

Fajfar, P. and Fischinger, M. (1988). "N2 – A method for nonlinear seismic analysis of regular buildings." *Proceedings of the 9th world conference in Earthquake Engineering.,* 111-116.

Fayaz R. Rofooei and M.R. Mirjalili. (2018). "Dynamic-based pushover analysis for one-way plan-asymmetric buildings." *Engineering Structures,* 163,332–346.

FEMA-356 (Federal Emergency Management Agency). (2000). "Prestandard and commentary for seismic rehabilitation of buildings." Washington DC.

Gh. Ghodrati Amiri1, B. Mohebi2 and N. Maddah, (2008). "Optimization of Damage Index in RC Structures Using Genetic Algorithm." *The 14th World Conference on Earthquake Engineering, Beijing., China.*

Gulkan P, Sozen MA. (1974). "Inelastic response of reinforced concrete structures to earthquake ground motions." *Journal of American Concrete Institute*., 71,601–609.

Hede Neena and Babu Narayan K.S, (2014). "Sensitivity of Pushover Curve to Material and Geometric Modelling- An Analytical Investigation." *Engineering Structures., Elsevier,* 91-97.

Hernandez-Montes E, Kwon O-S, Aschheim MA. (2004). "An energy-based formulation for first-and multiple-mode nonlinear static (pushover) analyses." *Journal of Earthquake Eng,* 8(1),69–88.

Hirde Suchita and Bhoite Dhanshri, (2013). "Effect of Modelling of Infill Walls on Performance of Multi Story RC Buildings." *International Journal of Civil Engineering and Technology.,* 4, 243-250.

Inel M, Ozmen H B. (2006). "Effects of plastic hinge properties in nonlinear analysis of reinforced concrete buildings." *Engineering Struc*tures., 28,1494–502.

Jiang Y, Li G, Yang D. (2010). "A modified approach of energy balance concept based multimode pushover analysis to estimate seismic demands for buildings." *Engineering Structure.*, 32(5),1272–83.

Jianmeng M, Changhai Z and Xie L. (2008). "An improved modal pushover analysis procedure for estimating seismic demands of structures." *Earthquake Engineering and Engineering Vibration.,* 7,25-31.

Kadid, A. and Boumrkik, (2008). "Pushover Analysis of Reinforced Concrete Frame Structure." *Asian Journal of Civil Engineering (Building and Housing),*75-83.

Kalkan E and Kunnath SK. (2006). “Adaptive modal combination for nonlinear static analysis of building structures.” *Journal of Structural Engineering.*, 132(11),1721-31.

Kalkan, E. and Kunnath, S.K. (2004). “Method of modal combinations for pushover analysis of buildings.” *13th World Conference on Earthquake Engineering, Vancouver., B.C.Canada.*

Kargahi Mohsen and James C. Anderson, (2004). “Structural Optimization for Seismic Design.” *13th World Conference on Earthquake Engineering., Vancouver, B.C., Canada,* 1-6.

Kaveh. A and Dadfar. B, (2008). “Optimum Seismic Design of Steel Moment Resisting Frames by Genetic Algorithms.” *Asian Journal of Civil Engineering (Building and Housing),*9(2), 107-129.

Kilar V, Fajfar P. (1997). “Simple Pushover Analysis of Asymmetric Buildings.” *Earthquake Engineering and Structural Dynamics*.,26(2),233-49.

Konuralp Girgin and Kutlu Darılmaz. (2007), “Seismic Response of Infilled Framed Buildings Using Pushover Analysis.” Department of Civil Engineering, Istanbul Technical University., 34469, Maslak, Istanbul, Turkey. 54(5).

Krawinkler Helmut and Seneviratana G.D.P.K, (1998). “Pros and Cons of Pushover Analysis of Seismic Performance Evaluation.” *Engineering structures.,* 20(4-6),452-464.

Kumar Pavan G V A, Dutt Ramesh, Kishore Vijaya K, Babu Harindha (2015). “Evaluation of Seismic Vulnerability of Multistoried Structures Using Pushover Analysis.” *IOSR-JMCE.,* 12,18-30.

Lakshmanan. N, (2006). “Seismic Evaluation and Retrofitting of Buildings and Structures.” *ISET Journal of Earthquake Technology.,* 43(1-2),31-48.

Lawson R.S., Vance V., Krawinkler H. (1994). “Nonlinear Static Pushover Analysis – Why, When and How?.” *Proc. 5th US Conf. on Earthquake Engineering., Chicago IL,* 1,283-292.

Leelataviwat S, Saewon W, Goel SC. (2009). “Application of energy balance concept in seismic evaluation of structures.” *J Struct Eng.,*135(2),113–21.

Liu Yang, Kuang J.S. and Qunxian Huang. (2018). “Extended spectrum-based pushover analysis for predicting earthquake induced forces in tall buildings.” *Engineering Structures.,* 167,351–362.

Liu Yang, Kuang J.S. and Qunxian Huang. (2018). “Modified spectrum-based pushover analysis for estimating seismic demand of dual wall-frame systems.” *Engineering Structures.,* 165,302–314.

Mahaney J.A., Paret T.F., Kehoe B.E., Freeman S.A. (1993). “The Capacity Spectrum Method for Evaluating Structural Response during the Loma Prieta Earthquake.” Earthquake Hazard Reduction in the Central and Eastern United States: A Time for Examination and Action, Proceedings of National Earthquake Conference, Earthquake Engineering Research Institute., Oakland, CA, U.S.A., pp. 501-510.

Manoukas G, Athanatopoulou A, Avramidis I. (2011). “Static pushover analysis based on an energy-equivalent SDOF system.” *Earthquake Spectra.*,27(1),89–105.

Mehdi Poursha and Esmaiel Talebi Samarin. (2015). “The modified and extended upper-bound (UB) pushover method for the multi-mode pushover analysis of unsymmetric-plan tall buildings.” *Soil Dynamics and Earthquake Engineering.,* 71-114–127.

Mehmet Inel., Tjen Tjhin and Mark A.Aschheim (2003). “The significance of lateral load pattern in pushover analysis.” *Fifth national conference on earthquake engineering., Istanbul, Turkey.*

Moghdam. A.S and Tso W.K, (2000). “Pushover Analysis for Asymmetric and Set Back Multi Story Buildings.” *12WCEE.,* 1-8.

Mouzzoun M, Moustachi O, Taleb A and Jalal S, (2013) “Seismic Performance Assessment of Reinforced Concrete Buildings Using Pushover Analysis”, *IOSR – JMCE.,* 5, 44-49.

Mwafy A.M and Elnashai A.S, (2001). “Static Pushover Versus Dynamic Collapse Analysis of RC Buildings.” *Journal of Engineering Structures.,* 23,407-424.

Naeim, F. (1999). “Ten commandants on push over analysis.” John A. Martin and Associates Publication., Los Angles.

Nie Jianguo., Qin Kai and Xiao Yan. (2006). “Pushover Analysis of seismic behavior of a concrete filled rectangular tabular frame structure.” *Tsinghua science and technology.,* 11,124-130.

Pan Xiaolan., Zhi Zheng and Zhenyu Wang. (2017). “Estimation of floor response spectra using modified modal pushover analysis.” *Soil Dynamics and Earthquake Engineering.,* 92,472–487.

Panyakapa Phaiboon, (2014). “Cyclic Pushover Analysis to Estimate Seismic Demands for Buildings.” *Engineering Structures., Elsevier,* 10-23.

Paspuleti Chaitanya. (2002). “Seismic analysis of an older reinforced concrete frame structure.” Master Thesis., Engineering University of Washington.

Peng. Li and Weijian. Yi, (2008). “Experimental and Analytical Studies on the Confinement Effect of Transverse Reinforcement in Concrete Columns Under Cycle Loading.*” The 14th World Conference on Earthquake Engineering., Beijing, China.*

Poluraju P, Durgabhavani T, Mounika K, Nageswari M, Soni Priya K, (2011). “Non-Linear Pushover Analysis of Flatslab Building by Using SAP2000.” *International Journal of Recent Technology and Engineering.*,1.

Poursha M, Khoshnoudian F, Moghadam A. (2011). “A consecutive modal pushover procedure for nonlinear static analysis of one-way unsymmetric-plan tall building structures.” *Engineering Struct.*,33(9),2417–34.

Poursha M, Khoshnoudian F, Moghadam A. (2014). “The extended consecutive modal pushover procedure for estimating the seismic demands of two-way unsymmetric plan tall buildings under influence of two horizontal components of ground motions." *Soil Dyn Earthquake Engineering.*,63,162–73.

Rafael A. Salgado and Serhan Guner. (2018). "A comparative study on nonlinear models for performance-based earthquake Engineering." *Engineering Structures.*, 172, 382–391.

Rama Raju K, Cinitha A and Nagesh R Iyer (2012). "Seismic Performance Evaluation of Existing RC Buildings Designed as Per Past Codes of Practice." *Indian Academy of Sciences.*, 37, 281-297.

Raut Nivedita N and Ambadkar Swati, (2013). "Pushover Analysis of Multistoried Building." *Global Journal of Researches in Engineering Civil and Structural Engineering.*, 13, 4-8.

Reyes JC, Chopra AK. (2011). "Evaluation of three-dimensional modal pushover analysis for unsymmetric-plan buildings subjected to two components of ground motion." *Earthquake Eng Struct Dyn.*, 40(13), 1475–94.

Reyes JC, Chopra AK. (2011). "Three-dimensional modal pushover analysis of buildings subjected to two components of ground motion, including its evaluation for tall buildings." *Earthquake Eng Struct Dyn.*,40(7),789–806.

Rofooei, F.R. (2007). "Adaptive pushover analysis." *Asian Journal of civil engineering (Building and Housing).*, 8(03),343-358.

Saedi Daryan A, Soleimani S, Ketabdari H. (2017). "A modal nonlinear static analysis method for assessment of structures under blast loading." *J Vib Control.*,

Saiidi,M and Sozen, M.A.(1981). "Simple Nonlinear Seismic Response of Reinforced Concrete Structures." *Journal of Structural Division.*, ASCE, 107(05), 937-953.

Sharma A, Reddy G. R, Vaze K. K. & Eligehausen R. (2013). "Pushover experiment and analysis of a full scale non-seismically detailed RC structure." *Engineering Structures.*, 46, 218–233.

Sharma A. (2008). "Experimental and analytical investigation on behavior of scaled down reinforced concrete framed structure under monotonic pushover loads." *Bhabha Atomic Research Centre.*

Sharma, A, Reddy G. R, Eligehausen R, & Vaze K. K. (2011). "Experimental and analytical investigation on seismic behavior of RC framed structure by pushover method." *Structural Engineering and Mechanics.*, 39(1), 125–145.

Shibata, A. and Sozen, M.A. (1976). "Substitute –Structure Method for Seismic Design in RC." *Journal of Structural Division.*, 102, 1-18.

Skokani Mathew J. and Hart Gary C. (2002). "Reliability of nonlinear static methods for the seismic performance prediction of steel frame building." *12weee.*, 1-8.

Soleimani S, Aziminejad A, Moghadam A. (2017). Extending the concept of energy-based pushover analysis to assess seismic demands of asymmetric-plan buildings." *Soil Dyn Earthquake Eng.*,93,29–41.

Soleimani Sahman., Aziminejad Armin, Moghadam A.S.. (2018). “Approximate two-component incremental dynamic analysis using a bidirectional energy-based pushover procedure.” *Engineering Structures.*, 157,86–95.

Takeda, T., Sozen, M.A. and Neilsoen, N.N (1970). “Reinforced Concrete Response to Simulated Earthquakes.” *Journal of Structural Division.*, 96(12), 2557-257.

Thapa M. (2009). “Sensitivity of pushover analysis to the methods of modeling.” Master Thesis., Department of Civil Engineering, National Institute of Technology Surathkal, India.

Tjhin T, Aschheim M, Hernández-Montes E. (2005) “Estimates of peak roof displacement using “equivalent” single degree of freedom systems.” *J Struct Eng.*,131(3),517–22.

Zou X.-K and Chan C. M. (2005). “Optimal seismic performance-based design of reinforced concrete buildings using nonlinear pushover analysis.” *Engineering Structures*., 27,1289–1302.

LIST OF PUBLICATIONS

1. Ravikumara H S, **Supriya R Kulkarni**, Babu Narayan K S, (2016). "The Effect of Order of Hinge Formation on Deformation Characteristics of a Propped Cantilever". ***International Journal of Scientific Development and Research***, Vol. 1, Issue. 5, 194-198.

2. **Supriya R. Kulkarni**, Ravikumara H. S, and Dr. K. S. Babu Narayan, (2016). "A Review on Pattern of Plastic Hinge Formation in Pushover Analysis". ***International Journal of Scientific Development and Research,*** Vol. 1, Issue. 5, 290-293

3. **Supriya R. Kulkarni**, Ravikumara H. S, and K. S. Babu Narayan, (2017). "Consequence of Hinge Formation Sequence on Pushover Analysis Results". ***Manipal J. Sci. Tech.***, vol.2 (1), 41-46.

4. **Supriya R Kulkarni** and K S Babu Narayan, (2018). "Influence of Quality Parameters on Quantities of Pushover Analysis Results". **Second International Conference on Architecture Materials and Construction Engineering**, Kerala, Jan 19-20.

5. **Supriya R Kulkarni**, Babu Narayan K S, (2018). "Pushover Analysis – Result Borders Due to Hinge Formation Orders". ***Structural Monitoring and Maintenance,*** **Techno press**., vol.5(2),173-178

6. **Supriya R Kulkarni**, Babu Narayan K S. "Stochastic Modeling Strategies for Enhancement of Pushover Analysis Predictions". ***Earthquakes and Structures.,*** **Techno press** (Under Review)

CURRICULUM VITAE

Name: Supriya Ravindra Kulkarni

Contact No: +91-9740991430

E-mail: supriya101@gmail.com

Date of Birth: 17-12-1989

Address: "Trivikram" Plot No 03

S.V. Road, Chidambar Nagar,

Tilakwadi, Belagavi-590006

Karnataka,

India.

Educational Qualifications

Degree	University	Year of Passing
M.Tech (Structural Engineering)	Visvesvaraya Technological University	2014
B.E. (Civil Engineering)	Visvesvaraya Technological University	2012
Higher Secondary	Govindram Seksaria Science College	2008
Class X	M V Herwadkar English Medium School	2006

Teaching Experience

Name of the Institute	Post Held	Period
Shri Vasantrao Potdar Polytechnic, Belagavi Karnataka India.	Guest Lecturer	Jun 2014 – Dec 2014

Research Publications

Description	International	National
Journal Papers	3	1
Conference Publications	-	1