SEISMIC PERFORMANCE AND VULNERABILITY OF IS CODE DESIGNED RC FRAME BUILDINGS

A DISSERTATION

Submitted in partial fulfillment of the requirements for the award of the degree
of
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in
EARTHQUAKE ENGINEERING
(With Specialization in Structural Dynamics)

By
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CANDIDATE'S DECLARATION

I hereby declare that the work that is being presented in this dissertation report entitled **"Seismic Performance and Vulnerability of IS Code Designed RC Frame Buildings"** in partial fulfillment of the requirements for the award of the Degree of "**Master of Technology**" in Earthquake Engineering with specialization in **Structural Dynamics**, submitted to the Department of Earthquake Engineering, IIT Roorkee is an authentic record of my own work carried out under the supervision of **Dr. Yogendra Singh,** Associate Professor, Department of Earthquake Engineering, IIT Roorkee, Roorkee, during academic session August 2007-June 2008. The matter embodied in this Dissertation report has not been submitted for the award of any other Degree or Diploma.

Date: 30/6/2008
Place: Roorkee

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CERTIFICATE

This to certify that the above statement made by the candidate is correct to the best of my knowledge.

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ABSTRACT

Significant developments have taken place in the last three decades in the field of Earthquake Resistant Design. Concepts of Capacity Design, Performance Based Design, and Displacement Based Design have been developed and are being adopted by many national codes. Indian standard seismic design practice is based on Force Based Design combined with some aspects of Capacity Design. This Dissertation examines the Seismic Performance and Vulnerability of RC frame buildings designed as per Indian Codes of Practice. Examples of conventional gravity load designed, and seismic load designed buildings, with and without ductile detailing are presented and their Performance and Seismic Vulnerability have been evaluated.

The seismic design codes, including the IS code, are traditionally based on Force Based Design approach, which incorporates effect of hysteretic damping through a Response Reduction Factor used for reducing the design loads. This factor includes the effect of Overstrength and Ductility. Two types of RC frame design – OMRF, without ductile detailing of reinforcement, and SMRF, with ductile detailing of reinforcement, are specified by the Indian codes. The efficacy of the Indian code design specifications are examined in this dissertation. It has been shown that the buildings designed and constructed properly for the gravity loads alone, have significant Overstrength available to sustain the MCE level seismic action corresponding to Zone-IV, without collapse. The RC frames designed as SMRF have much higher ductility but lesser strength as compared with OMRF and the seismic performance of the two designs has been observed to be comparable.

The seismic vulnerability of different design levels, as per IS codes has been studied and it has been shown that the deterministic framework of Performance Based Seismic Design, does not give complete insight into the expected performance and associated risks. It has been shown that SMRF design, with current design provisions of IS codes, has higher probability of damage as compared with OMRF. It is because of the maximum allowable drift limit in IS:1893, specified at design force level. It has been shown that the

probability of heavy and extensive damage can be significantly reduced by specifying the limits on total drift, as in case of IBC and EC-8.

Effect of URM infills on Dynamic Characteristics, Seismic Performance and Vulnerability of RC Frame Buildings has also been studied. The infills have been modeled as diagonal struts, with stiffness as defined in FEMA 356 and strength considered in various modes of failure. It has been shown that the infills have drastic effect on the seismic behaviour and performance of RC buildings. The stiffness and strength of buildings is remarkably increased, but, the ductility is drastically reduced due to URM infills. The overall performance and vulnerability of RC frame buildings is significantly deteriorated due to presence of URM infills.

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INTRODUCTION

1.1 GENERAL

Earthquake Resistant Design (ERD) of structures has developed greatly, since the initial ideas about ERD took shape in early twentieth century. Before the invention of accelerographs, it was difficult to quantify the fictitious inertial force acting during earthquake, and all the initial codes recommended a percentage of weight, about 10%, to be used as lateral load for analysis. After, measurement of ground accelerations, calculation of actual inertial forces acting on the structures was possible. The concept of Response Spectrum, which represents the maximum response amplitude of single degree of freedom systems with varying time periods (Otani, 2004) is one of the most important steps in the history of ERD. This concept was incorporated in the subsequent revisions of the codes. The other most important development, at philosophical level, was, understanding of ductility and hysteretic damping. Gradually, the ERD has developed significantly in the form of Capacity Design, Displacement Based Design, and Performance Based Design.

Till 1970s, various design philosophies like Working Stress Design, Ultimate Strength Design and Limit State Design were developed, in which individual members are proportioned for strength on the basis of internal forces computed from elastic analysis, only. However, the energy dissipation, because of the ductility present in the structure, enables us to design the structure for only 10-20% of forces corresponding to elastic response of the structure. It is expected that the structure will undergo large inelastic deformations without collapsing. The collapse can be avoided by facilitating plastic deformations in desirable ductile modes only. This can be achieved by designing the brittle modes to have strength higher than the ductile modes. This concept of "Capacity Design" has paved way for the development of a new design philosophy through which a desired strength hierarchy can be incorporated within the structural elements. For example, in case of frame buildings, the desirable mode of ductile failure can be achieved by weak beam - strong column design (Paulay & Pristley, 1992) that eventually dictates the modes of inelastic behaviour and failure. However, most of the codes, including Indian code, still follow the Force Based Design concept, in which the effect of ductility is considered indirectly in the form of a Response Reduction factor.

The aim of the present study is to examine the adequacy of the current provisions of IS codes for ERD of RC frame buildings, by studying, analytically, the seismic performance and vulnerability/fragility of IS code designed RC frame buildings.

1.2 FORCED-BASED DESIGN METHOD

Traditional code design practices are based on "Forced-Based Design", in which individual members are proportioned for strength so that the structure can sustain shocks of moderate intensities without structural damage and shocks of heavy intensities without total collapse, on the basis of internal forces computed from elastic analysis only. But, Pristley (1993, 2000 and 2003) and several other researchers have pointed out that force is a poor indicator of damage and there is no clear relationship between strength and damage. Hence, force can not be a sole criterion for design. Again assuming a flat response reduction factor for a class of building is not realistic because ductility depends on so many factors such as axial force, steel ratio, structural geometry, etc. Therefore, there is need for a design procedure, which is based on explicit estimates of the seismic performance.

1.3 DISPLACEMENT-BASED DESIGN METHOD

To overcome the flaws in Forced-Based Design, an alternative design philosophy named "Displacement-Based Design" was first introduced by Qi and Moehle(1991) which include translational displacement, rotation, strain, etc. in the basic design criteria. Displacement-Based Design is a useful and promising tool that enables designer to design a structure with predictable performance which is decided by owner, architect and structural designer.

Although the buildings designed using the current codes performed well in the recent earthquakes for Life Safety point of view, the damages incurred to were so high that either the building has lost its usage or the repair costs were very high. This is mainly because of the uncertainties in estimating the exact seismic capacity of the structure, particularly its ductility, variability and degradation in construction and intermediate steps. Further, the increasing cost of non-structural components and building contents is demanding better performance by the buildings during earthquakes. Performance Based Design (PBD) is an attempt by the structural engineers to evaluate the real strength and ductility of the structure and is an effective and useful tool for designing of structures for desired performance. In PBD, 'Performance Level' (acceptable damage condition) of the building after the design earthquake is decided based on the structural and non-structural performance levels of the structure and its elements (Ghobarah, 2001). A non-linear analysis is imperative for this purpose and after advent of Push-over Analysis non-linear analysis has become affordable by common designer. In Push-over analysis the magnitude of the lateral load is increased monotonically according to a predefined pattern. With the increase of the load, weak links and failure modes are identified.

1.4 PERFORMANCE BASED DESIGN METHOD

Performance Based Design is an important and useful method for displacement based design. The aim of Performance Based Design is to design a building for optimum cost keeping in mind its importance and desired Performance Level for specified hazard. In the conventional design practice, structural performance criteria is defined in terms of limits on member forces resulting from a prescribed level of earthquake loads whereas in Performance Based Design, structural criterions are expressed in terms of achieving a 'Performance Objective. A seismic Performance Objective has two essential parts – a damage state and a level of hazard. A Performance Objective represents a specific risk. With the recent development of analysis tools, it is possible to investigate buildings for multiple Performance Objectives. This approach provides the building owners and policy makers a framework for informed judgment and acceptability of various risks.

1.5 DESIGN PHILOSOPHY OF IS CODE

The primary purpose of all structures is to support self weight and gravity loads. But due to wind or earthquake, lateral force is generated in addition to self weight and gravity loads. To resist these additional lateral forces structure has to be designed with some special considerations along with gravity load resisting principles. For the present study IS: 456 (2000), IS: 1893-Part 1, (2002) and IS: 13920 (1993) guidelines and/or combination of these guidelines have been followed for different design levels. For example IS: 456 (2000) specification requirements are satisfied for gravity design level whereas for Ordinary Moment Resisting Frame (OMRF) IS: 456 (2000), IS: 1893-Part 1, (2002) and for Special Moment Resisting Frame (SMRF) IS: 1893-Part 1, IS: 13920 (1993) along with IS: 456 (2000), requirements have been satisfied.

1.5.1 Design Philosophy for Gravity Load

The gravity design philosophy as depicted in IS: 456 (2000) aims that the structure should remain serviceable and withstands all type of gravity loads liable to act throughout its life span. The serviceability requirements limit the deflection and cracking. The acceptable limit for safety and serviceability before failure of the structure is termed as 'Limit State'. There are mainly two types of limit states. Ultimate Limit State dealing with strength, overturning, sliding, buckling, fatigue fracture, etc. whereas serviceability limit state deals with discomfort to occupants due to excessive deflection, crack-width, vibration, etc. The structure should not reach a Limit State within its life span. The main focus of gravity load design philosophy is based on that Limit State only. The Limit State concept takes advantages of multiple factors of safety, in materials, and in loads that eventually provide adequate safety at service load as well as at ultimate load.

1.5.2 Design Philosophy for Seismic Load

The design philosophy adopted in the code (IS: 1893-2002) is to ensure that the structure should possess a minimum strength to withstand Design Basis Earthquake (DBE) which can reasonably be expected to occur at least once during the design life of the structure, without any significant structural damage, though some non-structural damage can be allowed. The structure should withstand the Maximum Considered Earthquake (MCE), the most severe earthquake effects, without collapse. Structures is expected to undergo large inelastic deformation without collapse, and hence can be designed for much lesser force than the actual forces that are expected to come during earthquake, as it is not economically feasible to have complete protection against all sizes of earthquake. Thus the basic criterions for earthquake resistant design are strength as well as ductility.

1.6 OBJECTIVES

The present study has been carried out with the following objectives:

1. To study the seismic performance of Medium-rise (4 storey) and High-rise (9 Storey) RC frame buildings, designed as per IS codes; and to examine the efficacy of the different design clauses of the IS codes, and their effect on seismic performance.

2. To study the seismic performance of IS code designed RC frame buildings in a probabilistic framework, by studying the effect of different design provisions on the seismic vulnerability of the buildings.

3. To study the effect of URM infills on the seismic performance and vulnerability of RC frame buildings.

1.7 SCOPE OF PRESENT STUDY

For this Dissertation work, parametric studies have been carried out for multistoried medium height (G+3) and high rise (G+8) RC frame buildings to assess the efficacy of the different clauses of the IS codes and to study the effect of different design considerations on the performance of the building and on their seismic vulnerability. Mainly five design levels are considered for this study such as gravity design, OMRF without capping on time period, OMRF with capping on time period, SMRF without capping on time period, and SMRF with capping on time period. The buildings have been designed for seismic zone IV. The three-dimensional bare frame has been modeled with lumped plasticity beam and column elements using SAP2000v10.05 software. To study the effect of URM infills on the RC frame, the infills have been modeled as strut elements considering in-plane stiffness of masonry as per FEMA 356. Out-of-plane stability of infill has not been considered. The effect of opening has also been ignored. The performance under DBE and MCE of the buildings for different design levels has been studied using Non-Linear Static Push-Over analysis and compared. Using the result of nonlinear static analysis seismic vulnerability/fragility curves are plotted and compared for different design levels. The variabilities defined in HAZUS have been used to plot the fragility curves.

1.8 ORGANIZATION OF DISSERTATION

In this Dissertation work, parametric study has been carried out on a set of multistoreyed RC frame buildings to assess the efficacy of the different provisions of the IS codes and to study the effect of different design considerations on the anticipated performance and seismic vulnerability of the buildings. Effect of URM infills on the performance and vulnerability of frame buildings is also studied for different design levels. This work is organized as follows:

CHAPTER 1 presents a brief outline of modern trends in the field of earthquake resistant design, developed in past three decades and scope of present study.

Seismic design philosophy and methodologies of Indian Standard Code of Practice is reviewed and compared with the design philosophy of other national seismic design codes in CHAPTER 2.

CHAPTER 3 describes the concepts of Performance Based Design vis-à-vis conventional Forced Based Design.

CHAPTER 4 consists of modelling, design, and analysis procedures for the generic RC buildings of different design levels, and comparison of their seismic performance.

Seismic vulnerability of RC buildings for different design levels is presented in CHAPTER 5 via. Fragility Curves and DPMs.

Effect of URM infills on the Seismic Performance and Vulnerability of RC frame buildings is reported in CHAPTER 6.

Finally, the conclusions of the study and recommendations for future work have been presented in CHAPTER 7.

PHILOSOPHY AND METHODOLOGY OF IS CODE DESIGN

2.1 GENERAL

Code design practices are traditionally based on Forced-Based Design (FBD) concept, in which individual members of the structure are proportioned for strength, so that it can sustain shocks of moderate intensities without structural damage and shocks of heavy intensities without total collapse, on the basis of internal forces computed from elastic analysis only. Design philosophy and salient features of Indian seismic design code are presented and compared with the philosophy of international seismic design codes. An attempt has been made to identify the lacunae of the design philosophy of the Indian standard.

2.2 SALIENT FEATURES OF INDIAN SEISMIC DESIGN CODES

The Indian code of practice for seismic design IS: 1893-2002 defines two levels of seismic hazard, namely Maximum Considered Earthquake (MCE) and Design Basis Earthquake (DBE). The DBE is considered as half of the MCE and structures are designed for DBE with partial load and material factors. Equivalent Static Load analysis, Modal Response Spectrum analysis and Linear Dynamic Time History analysis are prescribed to determine member design forces, based on the height and regularity of the building. The building is designed for a base shear calculated as

$$V_B = \frac{Z}{2}\frac{I}{R}\frac{S_a}{g}W \tag{2.1}$$

Where Zone factor (Z) represents the Effective Peak Ground Acceleration (EPGA), Importance Factor (I) and Response Reduction Factor (R) control the ductility demand, based on the anticipated ductility capacity and the post earthquake importance of the structure. For RC buildings two ductility classes based on reinforcement detailing are specified - Ordinary Moment Resisting Frames (OMRF) are assigned a reduction factor of 3 whereas Special Moment Resisting Frames (SMRF) are assigned a reduction factor equal to 5. In practice, the designers have a tendency to make flexible models of the buildings, as it results in lower design base shear due to enlarged time period. To safeguard against this error, the code has recommended a capping on the natural period used for base shear calculation. Empirical formulae for maximum periods to be used for base shear calculation have been provided in the code, and the design base shear is to be scaled up to the value calculated using the empirically obtained periods. Contrary to many other national codes, IS code specifies a limit (0.4%) for the interstorey drifts at the design (elastic) force level. In IBC and Euro Code, limits are specified for the total interstorey drift (including elastic and inelastic components). As different reduction factors (and hence different ductility demands) have been specified for OMRF and SMRF construction, it results in different limits on total drift. In other words, as per IS:1893, SMRF can be designed for about 1.67 times higher interstorey drift, as compared to OMRF.

IS:13920-1993 provides the specifications for ductile reinforcement detailing and Capacity Design. Interestingly, the provision for Strong Column-Weak Beam design to avoid column sway mechanism is somehow missing. Since, it is a widely recognized design criteria, the present study has been conducted ensuring Strong Column-Weak Beam in the design.

In Indian seismic design code, the inelastic effects are indirectly accounted for using a Response Reduction Factor based on some form of Equal Displacement and Equal Energy Principles. In the codal procedures, an explicit assessment of the anticipated performance of the structure is not made. To ensure the desired seismic performance, the national design codes exercise three types of controls in the design:

1. Control of Ductility Demand, using the effective Response Reduction Factor $\frac{I}{R}$, where I represents the Importance Factor and R represents the reduction factor for ductility and overstrength. Overstrength arises due to use of partial material and load safety factors and characteristic strength (grade) of material defined as 95% confidence values.

2. Control of minimum design base shear through the use capping on design natural period and/or flooring on the design base shear.

3. Control of flexibility through the maximum permissible interstorey drift.

The seismic performance of a building, designed according to the codal practices depends on the overall effect of the above controls and a several other provisions for design and detailing and the role of an individual control parameter is not explicit in the design.

Another emphasis of the code based design is enhancement of ductility by proper detailing and proportioning of members. Ductility can be enhanced by facilitating plastic deformations in desirable ductile modes only. This can be achieved by designing the brittle modes/members to have strength higher than the ductile modes. This concept of "Capacity Design" given by Park & Pauley (1974) has become integral part of the national design codes with desired strength hierarchy incorporated within the structural elements.

Preistley (1993, 2000 and 2003) and other researchers have clearly pointed out that force is a poor indicator of damage and there is no clear relationship between strength and damage. Hence, force can not be a sole criterion for design. Further, assuming a flat Response Reduction Factor for a class of buildings is not realistic because ductility depends on so many factors such as degree of redundancy, axial force, steel ratio, structural geometry, etc. Therefore, there is need for a design procedure, which is based on explicit estimates of the seismic performance.

CONCEPTS OF PERFORMANCE BASED DESIGN

3.1 GENERAL

The conventional method of design is based on the theory that strength capacity of structural members should be greater than the load demand on the members. This method suits for design against gravity and wind loads. For earthquake resistant design, a different design philosophy relying on ductile nonlinear behaviour is considered in which the main requirement is to ensure that the member deformation capacities are greater than the member deformation demands imposed by earthquakes. Performance Based Design (PBD) is multidisciplinary design approach with the aim to design a building with optimum cost keeping in mind its importance and desired performance level for specified hazard. In the conventional design practice, structural performance criteria is defined in terms of limits on member forces resulting from a prescribed level of earthquake loads, whereas in, PBD, design criteria are expressed in terms of achieving a performance objective. According to FEMA 356 , performance is measured in terms of the amount of damage sustained by the structure, when affected by earthquake ground motion, and the impacts of this damage on post-earthquake functionality of the structure.

3.2 SEISMIC HAZARD AND PERFORMANCE LEVELS

Before analysis and design of a building, it is important to estimate that to which level of ground shaking, the building is expected to be subjected during a future earthquake. The level of ground shaking at the site for a given earthquake or expected level of ground shaking at site is termed as Seismic Hazard. The seismic hazard at a site depends on the source and site parameters of the area and the probability of occurrence of earthquake. Obviously, the bigger earthquake events have lower probabilities of occurrence.

A performance level describes a predefined limiting damage condition for a given building and a given ground motion. The limiting condition is described by the physical damage within the building, the threat to life safety of the building's occupants created by the damage, and the post-earthquake serviceability of the building. Building Performance

Levels are a combination of a structural performance level and a nonstructural performance level. The definitions of building performance levels given by FEMA 356 are shown in the Table 3.1. In PDB structure can be designed for IO, LS, and CP performance levels.

Table 3.1 Damage Control and Building Performance Levels as per FEMA 356

Target Building Performance Levels	Overall Damage	General	Nonstructural Components
Operational Level	Very Light	No permanent drift in the structure. Structure substantially retains original strength and stiffness. Minor cracking of facades, partitions, and ceilings as well as structural elements. All systems important to normal operation are functional.	Negligible damage occurs. Power and other utilities are available, possibly from standby sources.
Immediate Occupancy Level	Light	No permanent drift in the structure. Structure substantially retains original strength and stiffness. Minor cracking of facades, partitions, and ceilings as well as structural elements. All systems important to normal operation are functional. Elevators may be restarted. Fire protection remains operable.	Equipments and contents are generally secure, but may not function properly due to mechanical failure or lack of utilities.
Life Safety Level	Moderate	Some permanent drift in the structure. Some residual strength and stiffness are still left in all stories. Gravity load bearing elements function properly. No out-of-plane failure of infill and tipping of parapet. Partition walls are damaged. Building may be beyond economical repair.	Falling hazards mitigated but architectural, mechanical and electrical systems may get damaged.
Collapse Prevention Level	Severe	Large permanent drift in the structure. Very little residual strength and stiffness are left, but load bearing columns and walls function. Infills and unbraced parapets failed or on the verge of failure. Building is near collapse.	Extensive Damage

In PBD, the real strength and ductility of the structure is evaluated and the Performance Level is controlled in terms of in-elastic deformations in different members, as in-elastic deformations are the best indicators of damage. A non-linear analysis is imperative for this purpose and advent of Push-over Analysis has been the most significant development to make it affordable by common designer. In Push-over analysis the magnitude of the lateral load, distributed according to a predefined pattern along the height of the building, is increased monotonically. With the increase of the load, weak links and failure modes are identified. This provides an insight into the non-linear behaviour, and reasonably accurate estimation of the seismic performance of the building, with much reduced computational effort. The details of the procedure and various limit states have been well documented by FEMA.

3.3 ANALYSIS PROCEDURES

Performance Based Design requires estimation of level of damage in individual members of the building. This needs a detailed analysis of the building structure for the estimated earthquake hazard. The analysis procedures are classified (Fig: 3.1) based on the Modelling – Linear and Non Linear Procedures; and based on the nature of earthquake forces considered – Static and Dynamic Procedures. Based on these two classifications for types of analysis procedures have been identified in literature:

- Linear Static Procedure (LSP)
- Linear Dynamic Procedure (LDP)
- Non-linear Static Procedure (NSP)
- Non-linear Dynamic Procedure (NDP)

Linear procedures are appropriate when the expected level of non-linearity is low. The static procedures are considered to be adequate if the contribution of higher modes is not significant. If the contribution of the first mode is more than 75%, the static analysis is considered to be adequate. Dynamic analysis is necessary in the following cases:

- Tall buildings, having significant contribution of higher modes. Buildings more than 20 storeys should be analyzed using dynamic analysis.

- Buildings with torsional irregularities
- Buildings with non-orthogonal systems

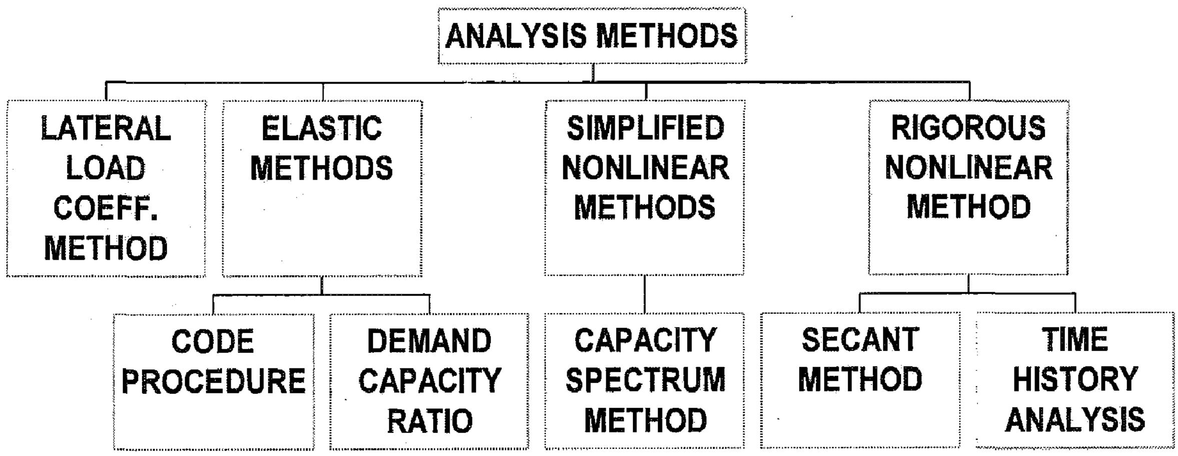

Fig: 3.1 Available Analysis Methods

3.4 MATHEMATICAL MODELING FOR ANALYSIS

The linear modeling basically consists of simulation of the relative stiffness of different components of a building since the forces are distributed among the different components in the ratio of stiffnesses. In non-linear analysis, in addition to the stiffness, strength and ductility of individual members is also simulated. In addition to these, the strength properties in different actions, including the non-linear load-deformation curves, are also modeled for each member. For skeletal frame members, to approaches are available for non-linear modelling:

I) *Distributed Plasticity Approach :* In this approach, it is assumed that yielding is distributed over a length of member. The structural characteristics of the member are calculated by assuming a displace shape of the member axis, with internal forces calculated at various sections from the resulting curvatures and axial strains. The member stiffness is then determined by integrating along the member.

II) *Lumped Plasticity Approach :* In this approach, it is assumed that yielding takes only at generalized plastic hinges of zero length, and the member between these hinges is assumed to be linearly elastic. Multidimensional action-deformation relationships are specified for the hinges, in terms of moment and axial force actions and deformations such as hinge rotations and axial extensions.

Lumped plasticity models are particularly suitable for analysis of building frames under seismic loading, because plastic action in such a structure is usually confined to small lengths at beam and column ends.

3.5 NON-LINEAR ANALYSIS PROCEDURES

In earthquake resistant design structures are designed to go into nonlinear range during a severe earthquake before collapse. The earthquake force calculated for a linear structure is reduced by a reduction factor based on available ductility ratio and over strength in the structure. It has been seen in the past earthquakes that this criteria is generally adequate for the normal type of structures, but, for structures with some irregularity or structures required to satisfy a particular performance level, this criteria is not sufficient and a nonlinear analysis of the building is required.

The most basic nonlinear analysis procedure is the complete nonlinear time history analysis or the Non-Linear Dynamic Procedure (NDP). Difficulty in selection of design time history, and complexity involved in the procedure makes it impractical for general uses. FEMA 356and ATC 40 present some simplified nonlinear analysis methods, which can be used easily for practical design purpose. The method is known as Nonlinear Static Procedure (NSP) or Nonlinear Pushover Analysis.

3.5.1 Displacement Coefficient Method (DCM) As Per FEMA-356/FEMA-440

For this present study, Displacement Coefficient Method (DCM) has been used for the purpose of nonlinear analysis. This method is based on the estimation of a 'Target Displacement'. The Target Displacement is the probable displacement of the building estimated using the Capacity Curve of the building and some fundamental concepts of structural dynamics. The target displacement can be obtained as,

$$\delta_t = C_0 C_1 C_2 C_3 S_a \frac{T_e}{4\pi^2} g \tag{3.1}$$

Where,

C_0 = Modification factor to relate spectral displacement of an equivalent SDOF system to the roof displacement of the building MDOF system calculated using one of the following procedures:

- The first modal participation factor at the level of the control node;
- The modal participation factor at the level of the control node calculated using a shape vector corresponding to the deflected shape of the building at the target displacement. This procedure shall be used if the adaptive load pattern is used;

C_1 = Modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response:

$= 1.0$ for $T_e \geq T_S$

$= [1.0 + (R-1)T_s / T_e]/R \quad for \quad T_e < T_s$

T_e = Effective fundamental period of the building in the direction under consideration, sec.

T_s = Characteristic period of the response spectrum, defined as the period associated with the transition from the constant acceleration segment of the spectrum to the constant velocity segment of the spectrum.

R = Ratio of elastic strength demand to calculated yield strength coefficient calculated by Equation (3.2).

$C_2 =$ Modification factor to represent the effect of pinched hysteretic shape, stiffness degradation and strength deterioration on maximum displacement response. Values of for different framing systems and Structural Performance Levels shall be obtained from FEMA-356. Alternatively, use of $C_2 = 1.0$ shall be permitted for nonlinear procedures.

$C_3 =$ Modification factor to represent increased displacements due to dynamic P-Δ effects. For buildings with positive post-yield stiffness, shall be set equal to 1.0.

$S_a =$ Response spectrum acceleration, at the effective fundamental period and damping ratio of the building in the direction under consideration, *g*.

The strength ratio *R* shall be calculated in accordance with Equation (3.2):

$$R = \frac{S_a}{V_y / W} C_m \tag{3.2}$$

Where,

$V_y =$ Yield strength calculated using results of the NSP for the idealized nonlinear force displacement curve developed for the building in accordance with procedure of FEMA 356.

$W =$ Effective seismic weight.

$C_m =$ Effective mass factor, C_m can be taken as the effective model mass calculated for the fundamental mode using an Eigenvalue analysis.

SEISMIC PERFORMANCE OF RC FRAME BUILDINGS

4.1 GENERAL

A comparison of nonlinear characteristics, seismic performance and yield pattern at ultimate point of RC buildings of different design levels have been studied in this Chapter. Effect of various seismic design specifications on Performance Levels of different design level buildings have been studied and compared. Demand Ductility, Capacity Ductility, Overstrength Factors and Effective Reduction Factors are computed using Capacity Spectrum and are compared.

4.2 DESIGN OF GENERIC BUILDINGS

The basic structural configuration used for these case studies are multistoried medium height (G+3) and high rise (G+8) RC buildings (Fig.4.1). This plan is symmetric in longitudinal direction and slightly asymmetric in transverse direction representing slight torsion due to plan irregularity. There is large variation in number of indeterminacy in the two directions as the building has 8 bays in longitudinal direction, whereas it has only 3 bays in transverse direction. The storey height has been considered as 3.3m with Ground level height being 1.5m. The corridor possesses no transverse beams (only the slab runs over the corridor). The building has been assumed to be situated in Seismic Zone – IV. For design, M20 concrete and Fe415 steel have been used and optimum sizes of beams and columns have been obtained. The slab thickness has been assumed as 150mm and a uniform light weight partition of $0.5\ kN/m^2$ has been considered respectively. The design details are tabulated in Table 4.1.

The Dead Load (DL) and Live load (LL) have been calculated using Part 1 and Part 2 of IS 875: 1987, respectively. Seismic Design has been performed as per IS: 1893 (Part 1): 2002, considering R equal to 3 and 5 for OMRF and SMRF, respectively.

The Dead Load (DL) and Live loads (LL) have been calculated using Part 1 and Part 2 of IS 875: 1987 respectively. For seismic loading, Design Basis Earthquake (DBE) hazard has been used as per IS: 1893 (Part 1): 2002. Seismic Design has been performed as per

IS: 1893 (Part 1): 2002, considering *R* equal to 3 and 5 for OMRF and SMRF corresponding to conforming and non-conforming (FEMA-356) transverse reinforcement, respectively.

Table 4.1 Description of Buildings

No. of Stories	G+3, G+8
Seismic Zone	IV
Soil Type	Hard soil
Importance Factor	1
Grade of Concrete	M20
Grade of Steel	Fe415
Density of RC	25 kN/m^3
Live load at corridor	4 kN/m^2
Live load at rooms	3 kN/m^2

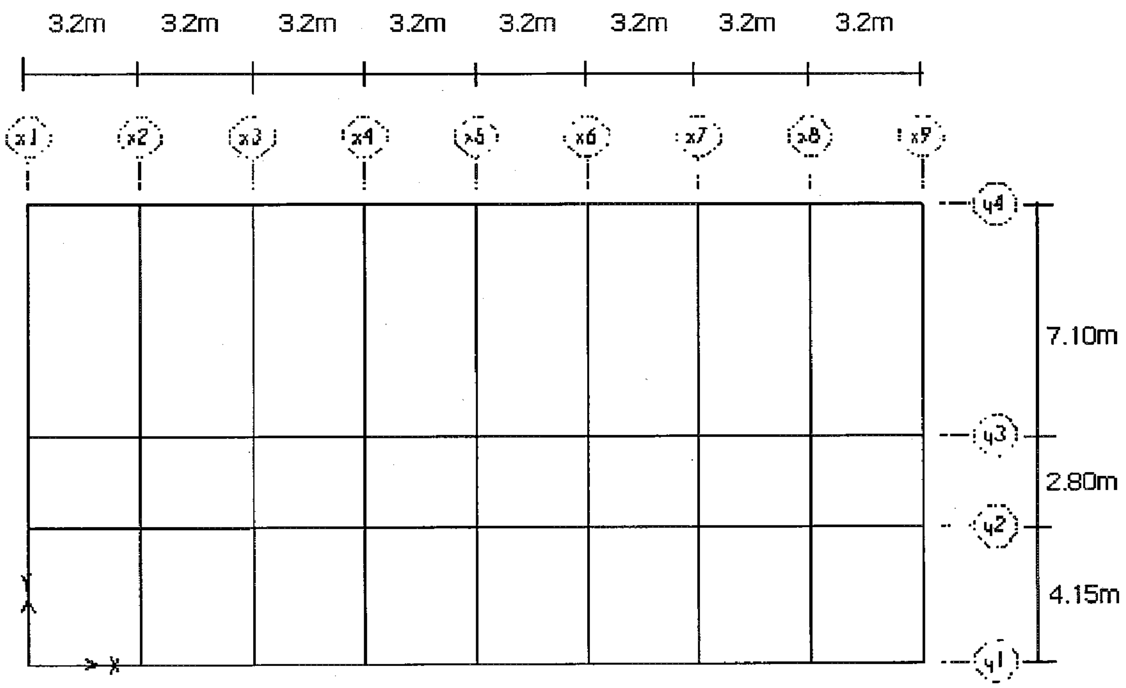

Fig. 4.1 Plan of Building

A response spectrum for rock has been adopted for the dynamic analysis of the building. The seismic weight considered for the analysis and Load combinations as per IS: 1893 (Part1 2002) have been used for the analysis and the worst load combination has been used for the subsequent design. The three-dimensional bare frame has been modeled using SAP2000v10.05 software for its Linear Analysis and design. The effect of torsion due to plan irregularity has been automatically taken care of. The slab has been modeled as a rigid diaphragm. The Linear analysis have been carried out for the bare frame with $R = 3$ and $R=5$ and limit state design is done for gravity loading and Zone IV for the same.

4.2.1 Design Levels

Mainly five design levels have considered for this study. In 'Gravity Design' level building has been designed only for gravity loads and no consideration has been given for seismic forces. Although, not permitted by the IS code in Seismic Zone IV, the most common type of design practice followed in India is OMRF, which has been considered with and without period capping, in the present study. Similary, SMRF has also been considered with and without period capping. Preliminary sizes of beams have been calculated based on deflection criterion as per IS: 456 (2000). The minimum and maximum reinforcement criteria of IS:456 and IS:1893 have been satisfied. For the purpose of comparison, buildings with and without satisfying the maximum drift limit as per IS: 1893 (2002) have also been considered. To study the effect of unequal inelastic drift limits in case OMRF and SMRF, a special case of SMRF with inelastic drift limit equalized to that of OMRF has also been considered.

4.3 NONLINEAR ANALYSIS

Nonlinear space frame models of the designed buildings were developed in SAP2000 Nonlinear software. Lumped plasticity models with hinge properties as defined in FEMA 356 (Tables 6-7 & 6-8) have been used. Confirming, 'C' and Non-confirming, 'NC' transverse reinforcement has been considered for OMRF and SMRF, respectively, to assign the plastic rotations for beams and columns. Three types of lateral load distributions: (i) proportional to first mode in the respective direction, (ii) parabolic distribution as per IS:1893, and (iii) uniform distribution have been considered for Pushover analysis. It has been observed that these result in only marginal difference in the capacity spectra and the results for parabolic distribution as per IS code are being presented here.

4.4 COMPARISION OF CAPACITY CURVES

Figs. 4.2-4.5 show the comparison of Capacity Curves and Performance Points for 4-storey and 9-storey buildings designed for gravity loads and as SMRF, as per relevant IS codes. Effect of ductile detailing (SMRF) as per IS: 13920 (1993) over Ordinary Moment Resisting Frame (OMRF) with respect to strength, ductility and enhancement of performance for 4-storey and 9-storey building is shown through Figs. 4.6-4.9. In practice, the designers have a tendency to develop flexible models of the buildings. The consequences of this general practice are also examined and effect of it on the behaviour of the building is presented through Figs. 4.10-4.13. In the Figs. The black dot (•) represents the Performance Point for DBE and black triangle (▲) represents the Performance Point for MCE; the three crosses (+) represent IO, LS and CP Performance Levels, consecutively.

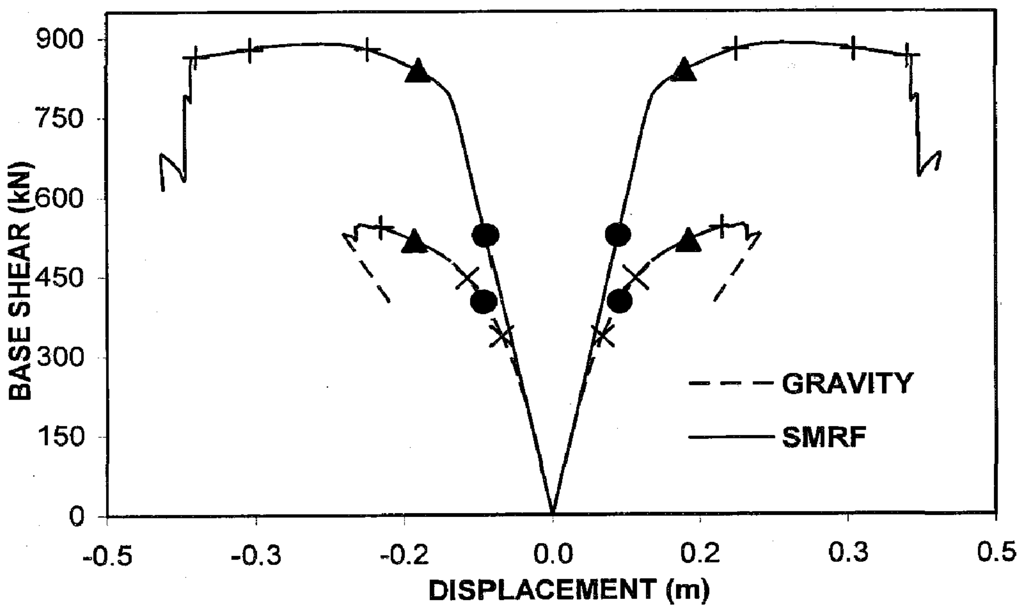

Fig. 4.2 Capacity Curves for 4 - Story Building Designed for Gravity and as SMRF, as per Relevant IS Codes; in Longitudinal Direction

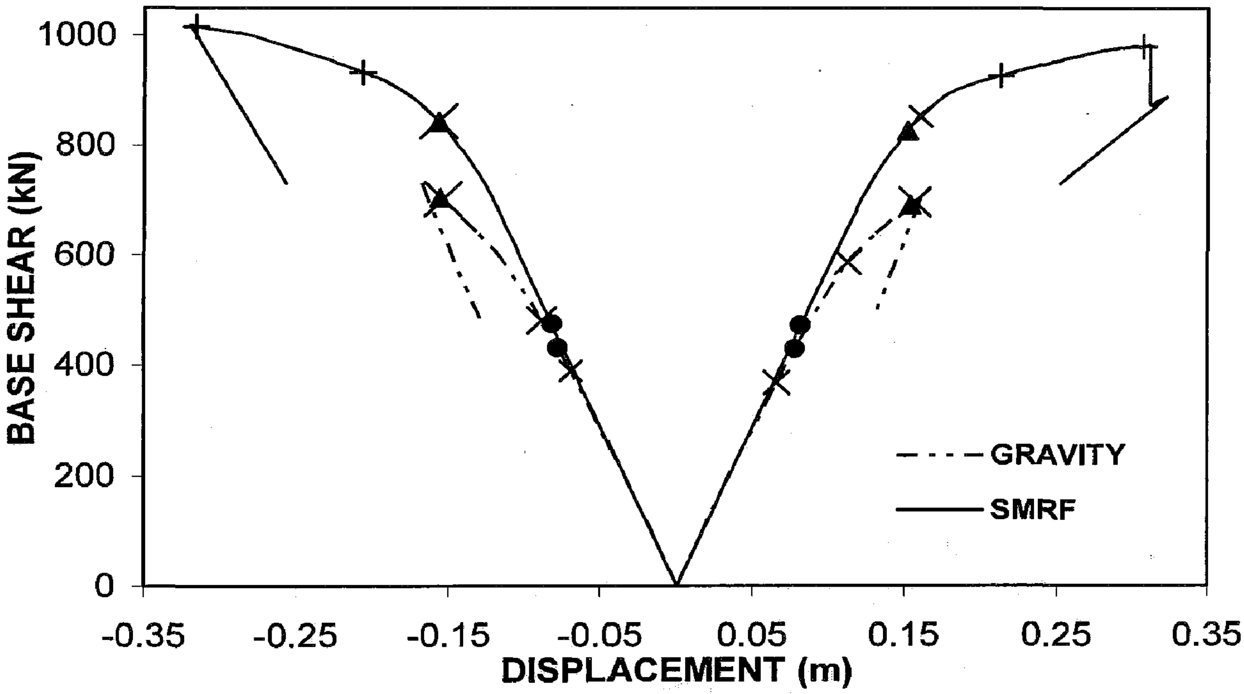

Fig. 4.3 Capacity Curves for 4 - Story Building Designed for Gravity and as SMRF, as per Relevant IS Codes; in Transverse Direction

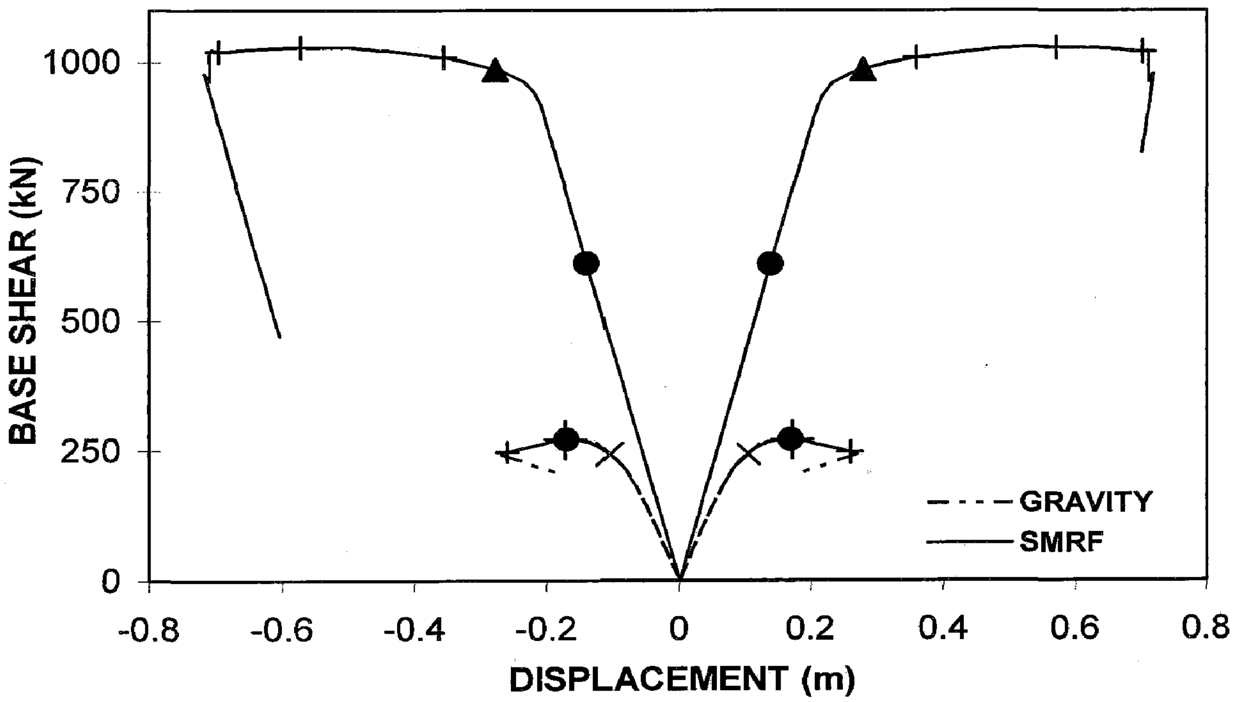

Fig. 4.4 Capacity Curves for 9 - Story Building Designed for Gravity and as SMRF, as per Relevant IS Codes; in Longitudinal Direction

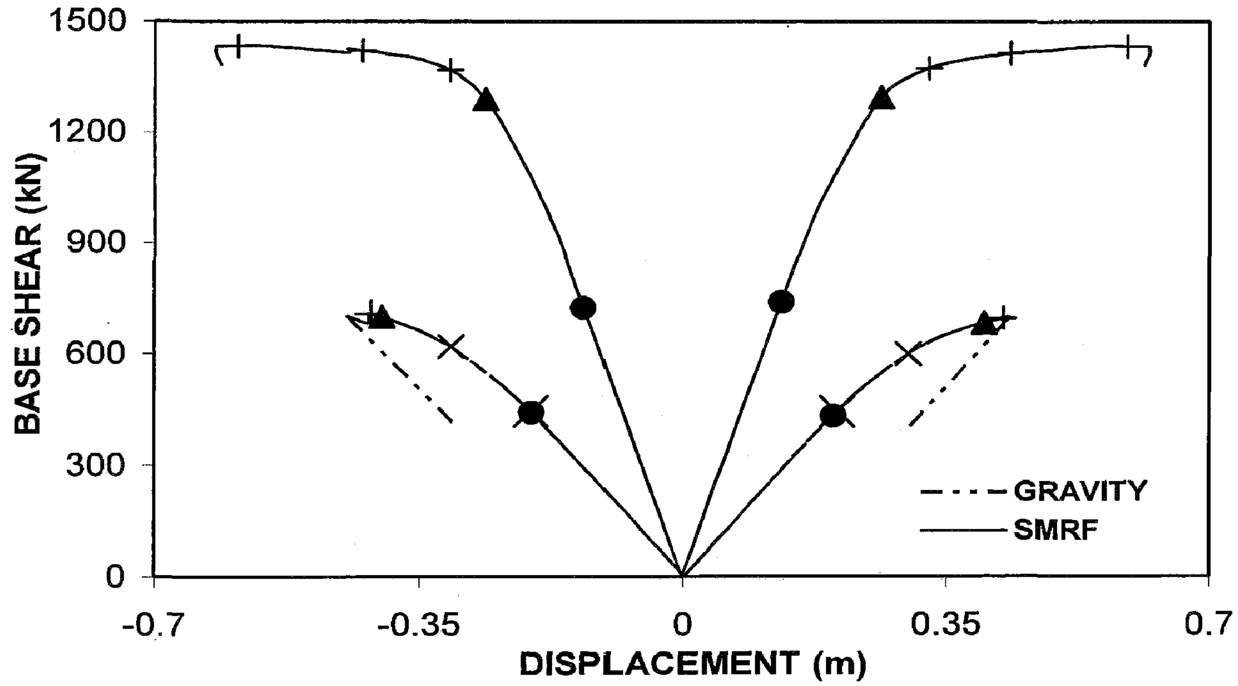

Fig. 4.5 Capacity Curves for 9 - Story Building Designed for Gravity and as SMRF, as per Relevant IS Codes; in Transverse Direction

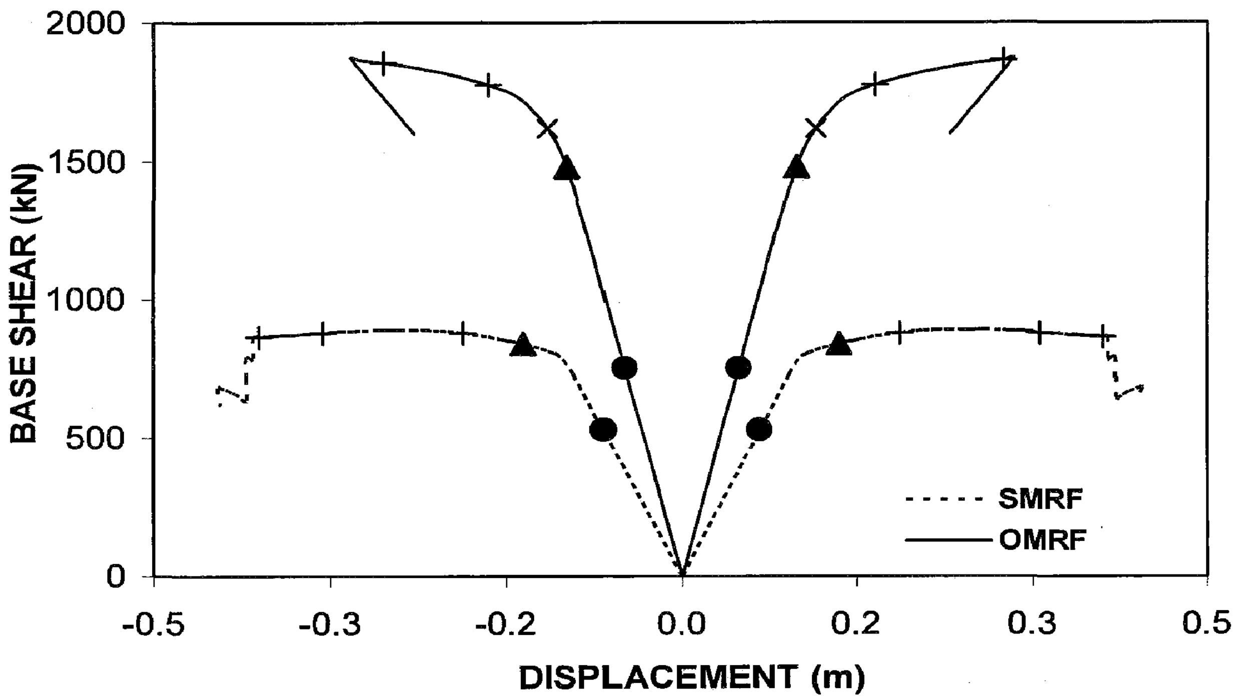

Fig. 4.6 Capacity Curves for 4 - Story Building Designed as OMRF and SMRF, as per Relevant IS Codes; in Longitudinal Direction

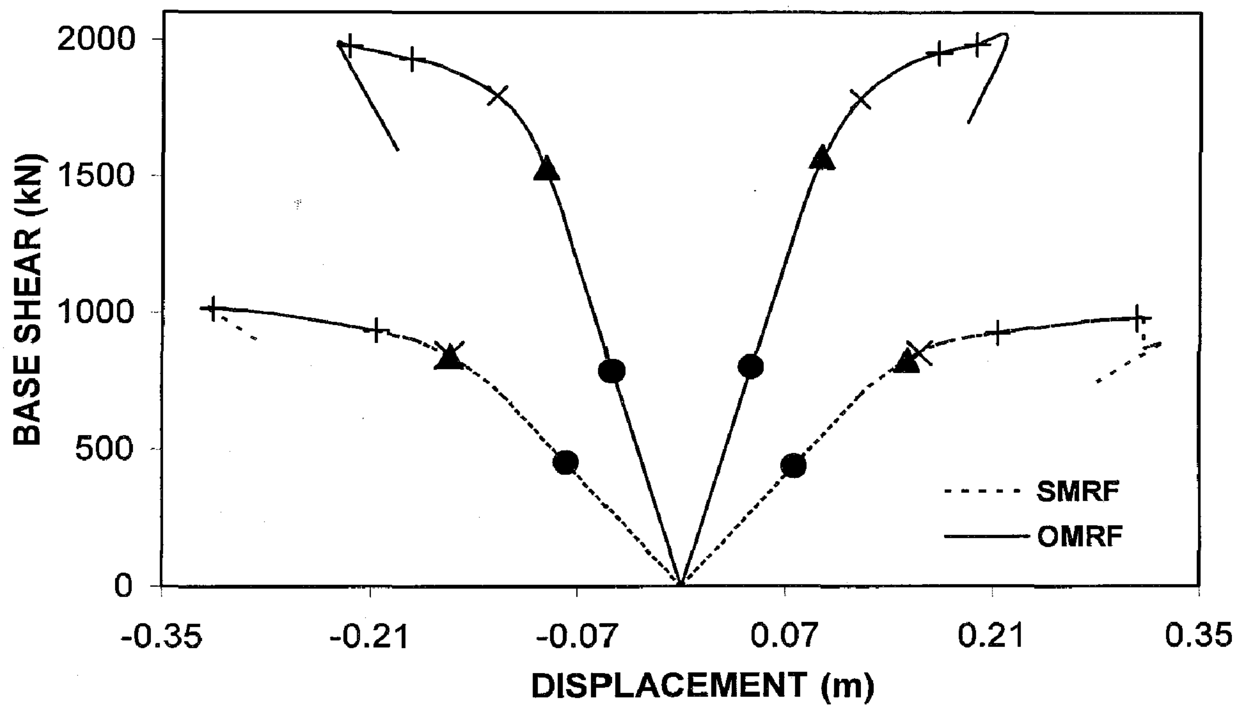

Fig. 4.7 Capacity Curves for 4 - Story Building Designed as OMRF and SMRF, as Per Relevant IS Codes; in Transverse Direction

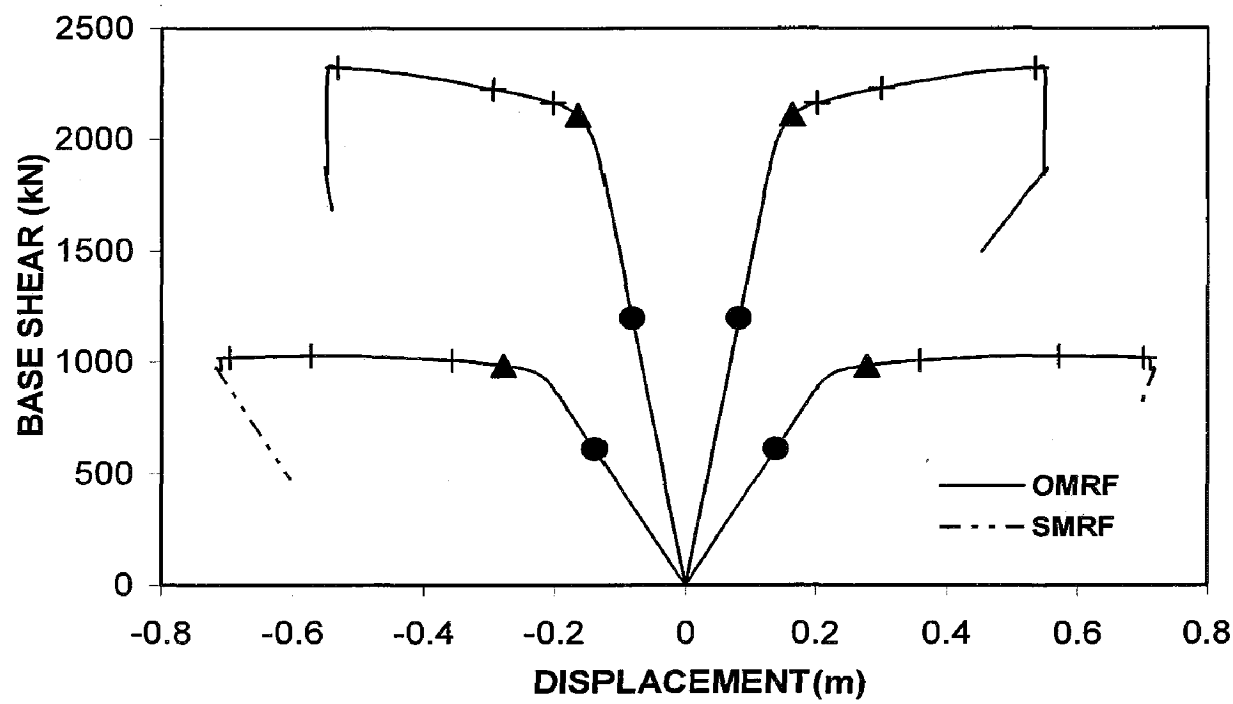

Fig. 4.8 Capacity Curves for 9 - Story Building Designed as OMRF and SMRF, as Per Relevant IS Codes; in Longitudinal Direction

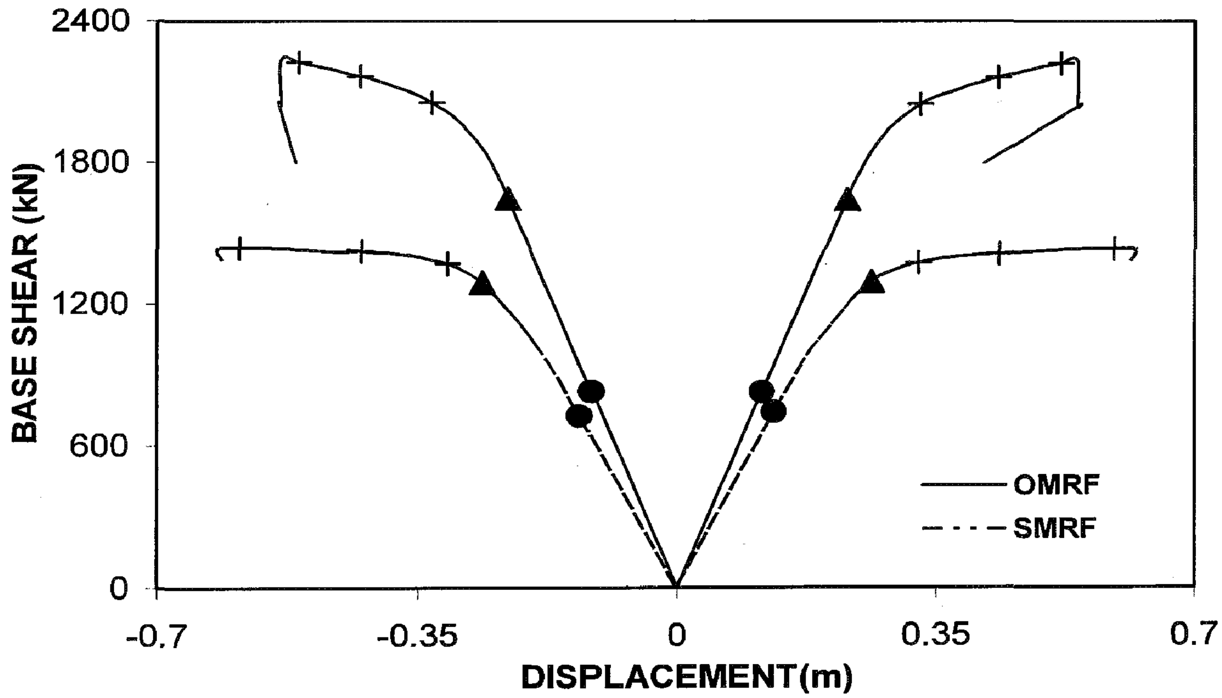

Fig. 4.9 Capacity Curves for 9 - Story Building Designed as OMRF and SMRF, as Per Relevant IS Codes; in Transverse Direction

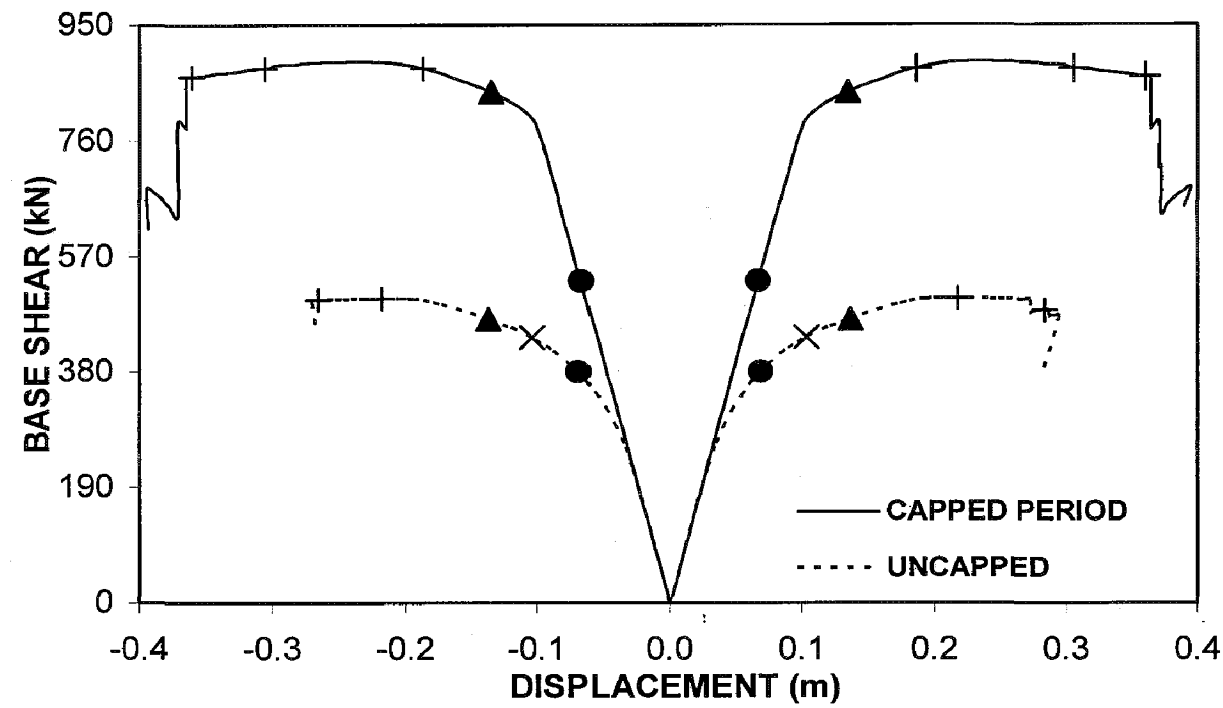

Fig. 4.10 Capacity Curves of 4 - Story Building in Longitudinal Direction, Showing Effect of Capping on Time Period

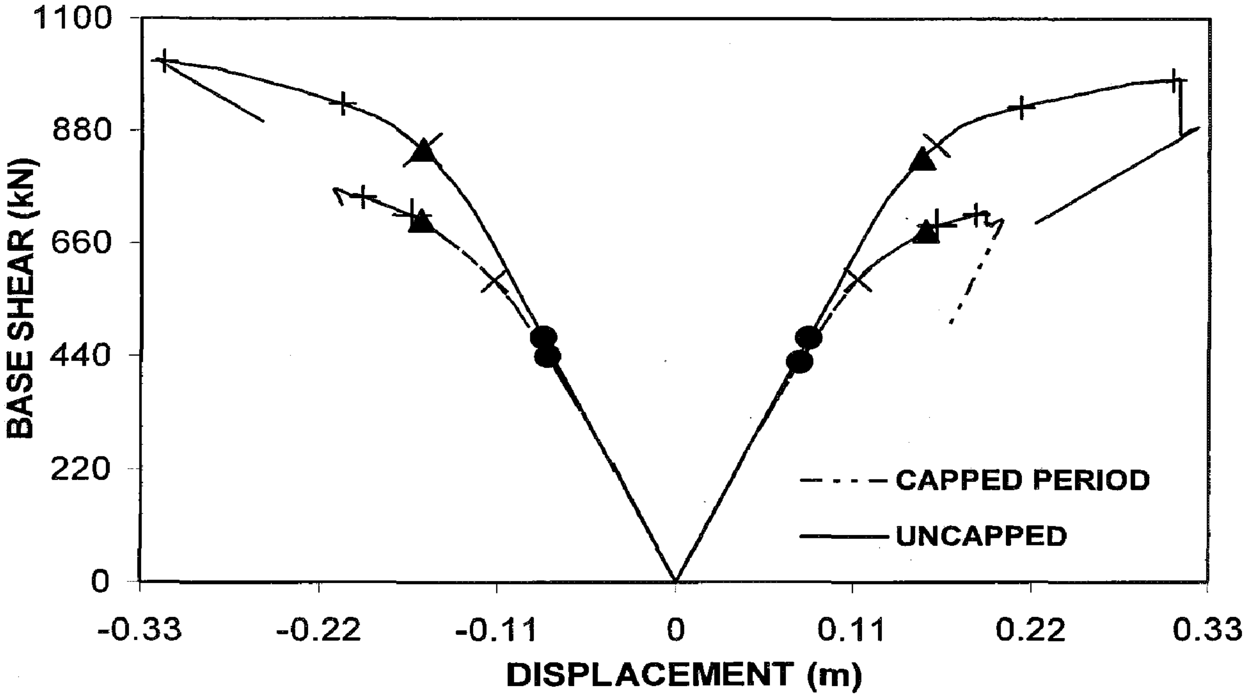

Fig. 4.11 Capacity Curves of 4 - Story Building in Transverse Direction, Showing Effect of Capping on Time Period

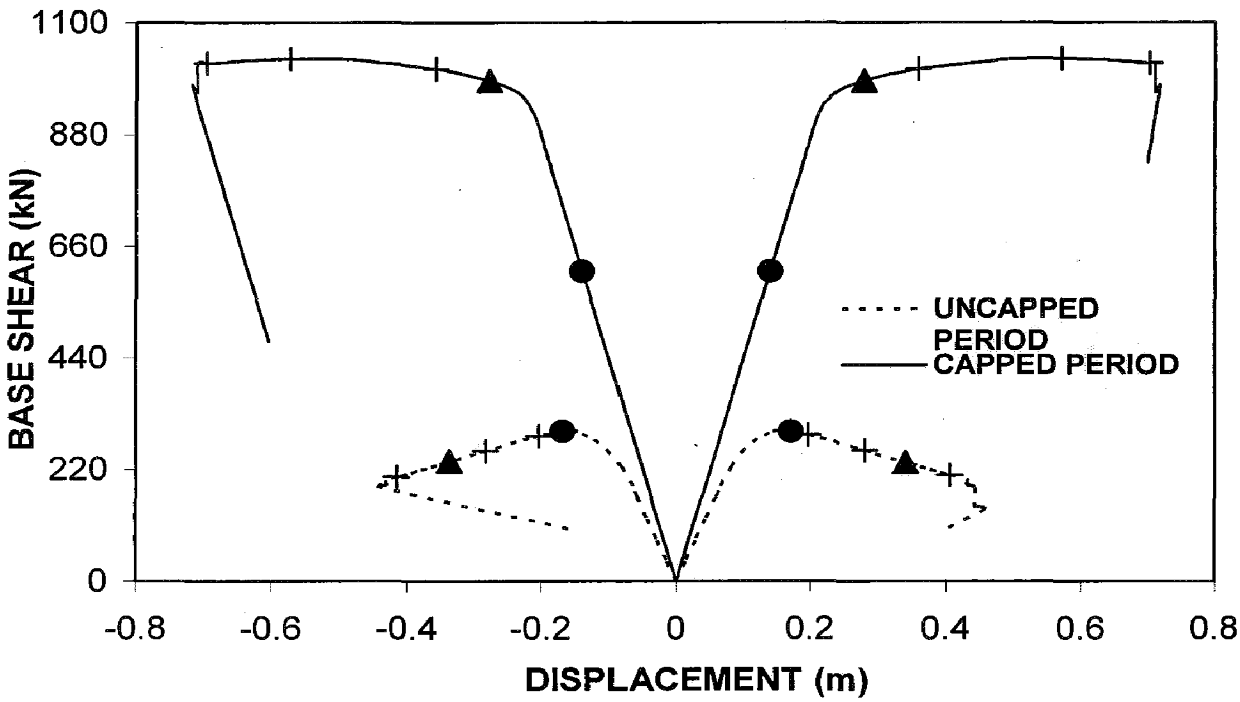

Fig. 4.12 Capacity Curves of 9 - Story Building in Longitudinal Direction, Showing Effect of Capping on Time Period

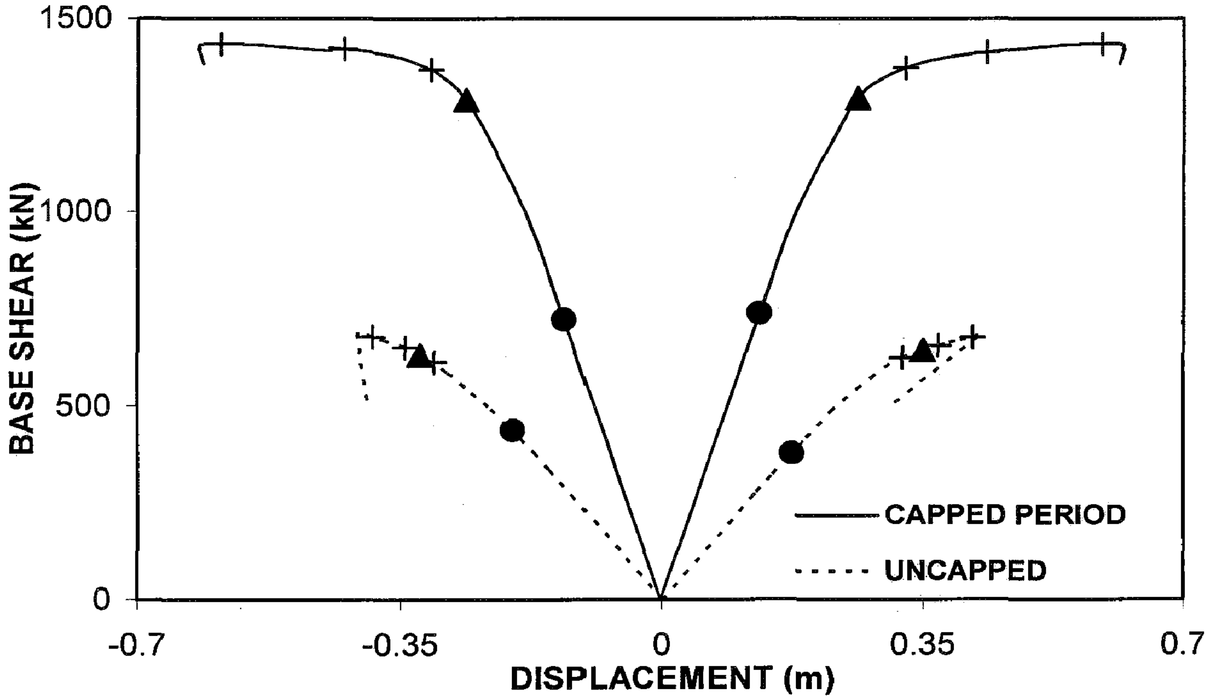

Fig. 4.13 Capacity Curves of 4 - Story Building in Transverse Direction, Showing Effect of Capping on Time Period

4.5 YIELD PATTERNS

A study of yield patterns of buildings designed different design levels has been studied to understand the inelastic behaviour under monotonically applied incremental static load in different directions.

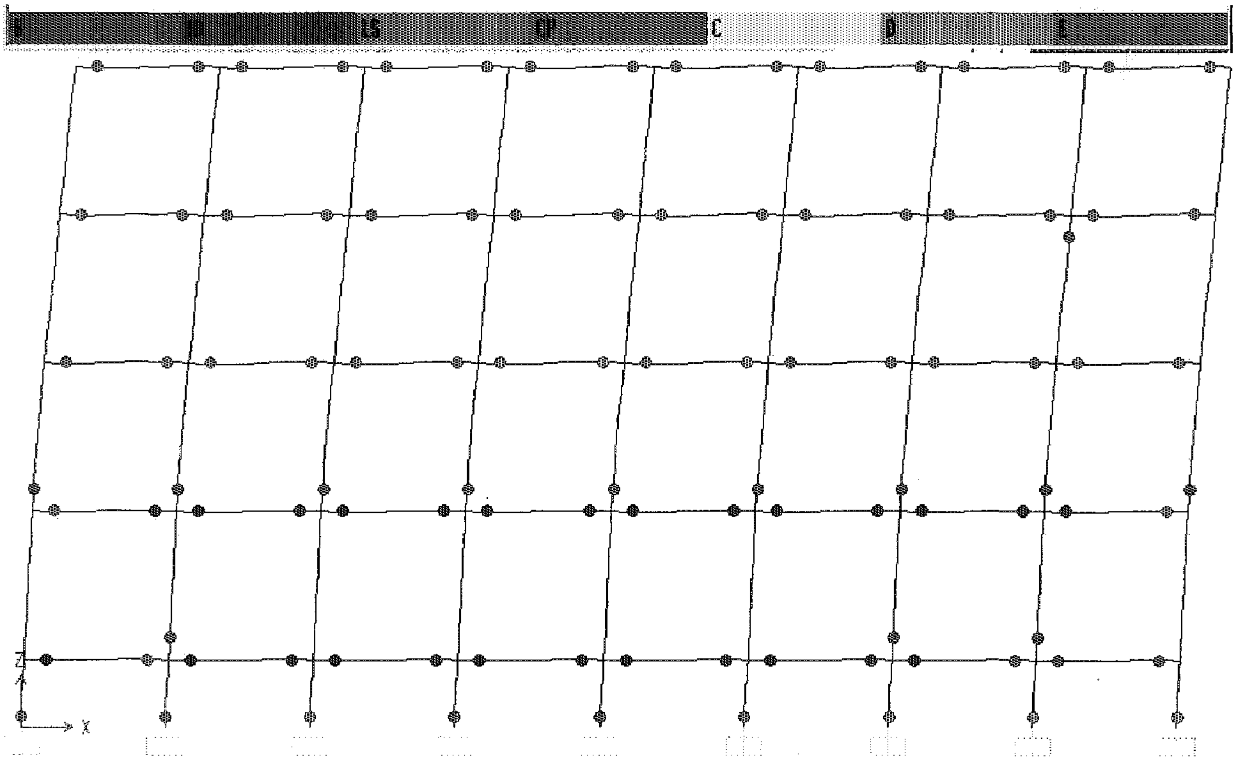

Fig. 4.14 Yield Pattern at Ultimate Point for 4-Storey Building Designed for Gravity in Longitudinal Direction

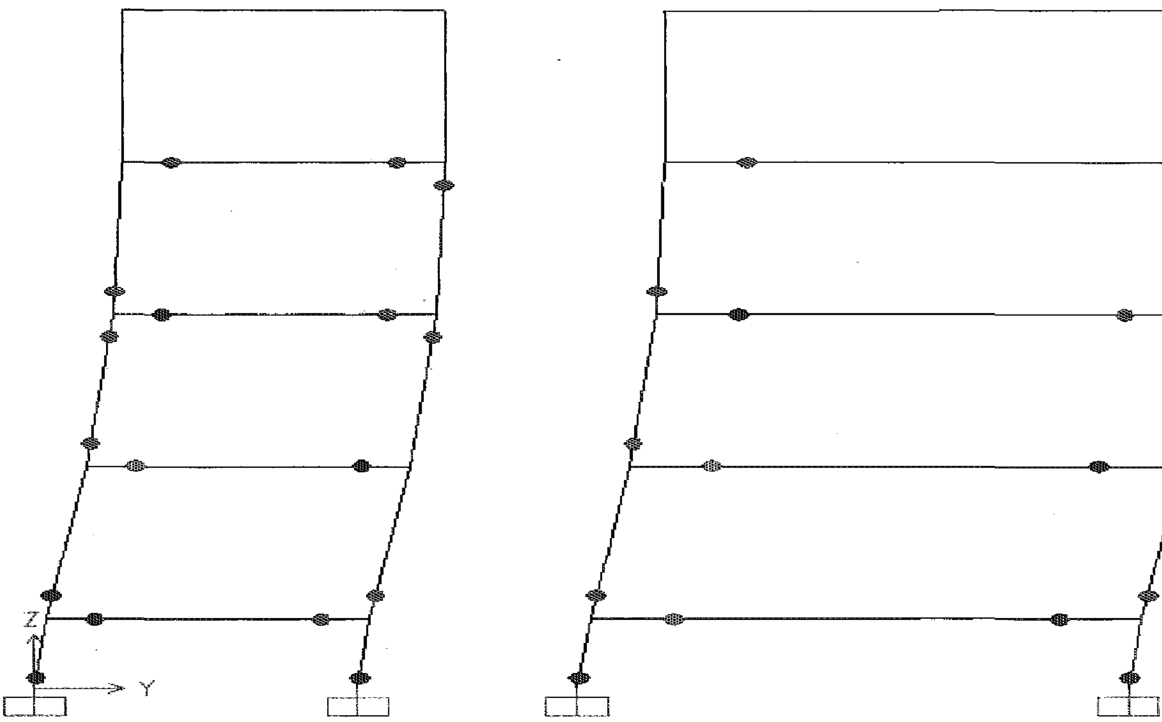

Fig. 4.15 Yield Pattern at Ultimate Point for 4-Storey Building Designed for Gravity in Transverse Direction

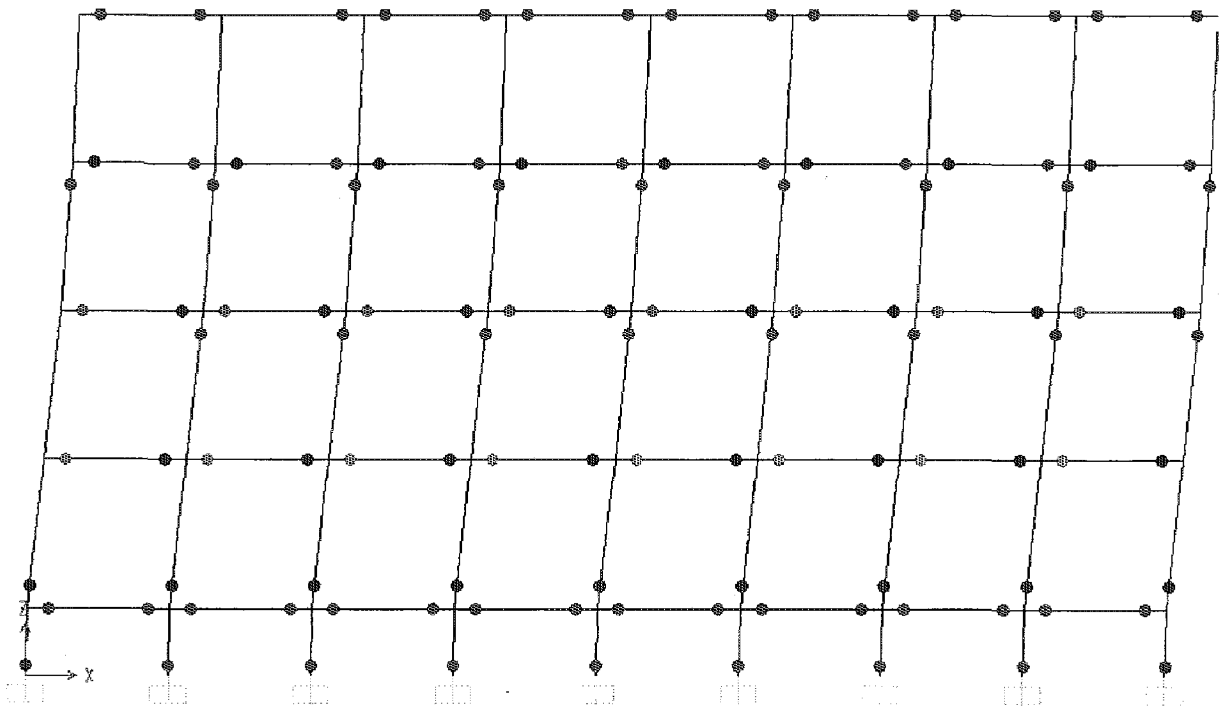

Fig. 4.16 Yield Pattern at Ultimate Point for 4-Storey Building Designed as OMRF in Longitudinal Direction

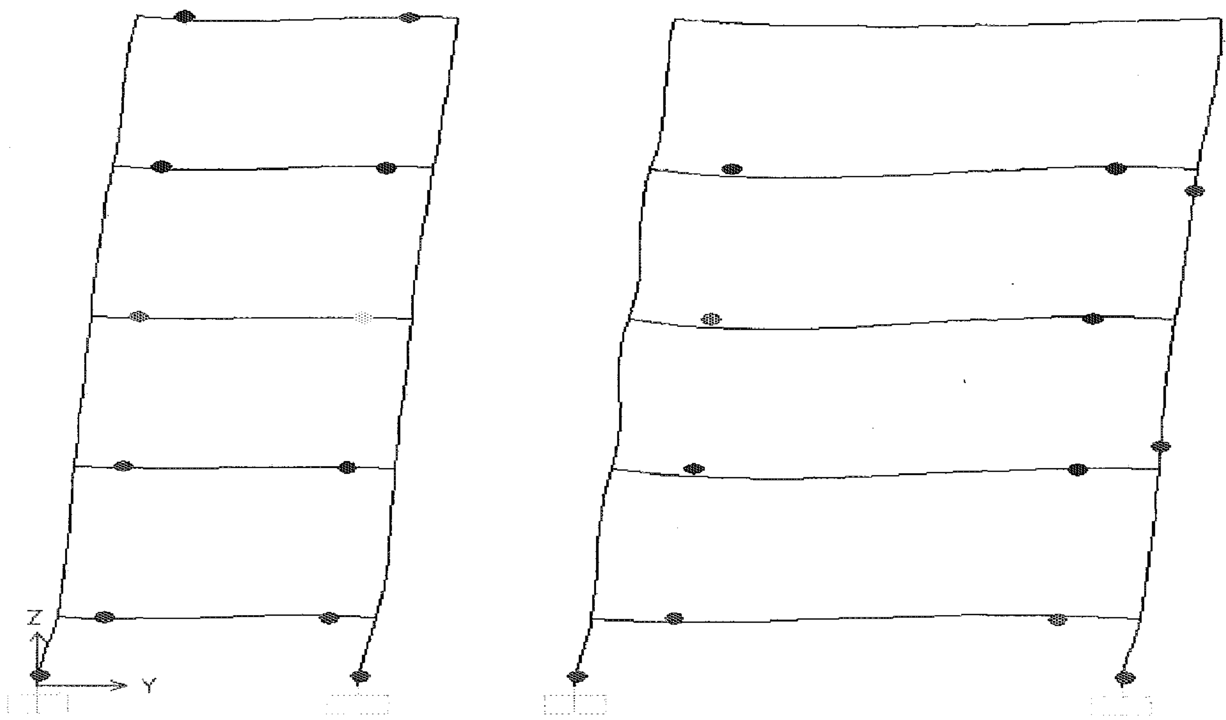

Fig. 4.17 Yield Pattern at Ultimate Point for 4-Storey Building Designed as OMRF in Transverse Direction

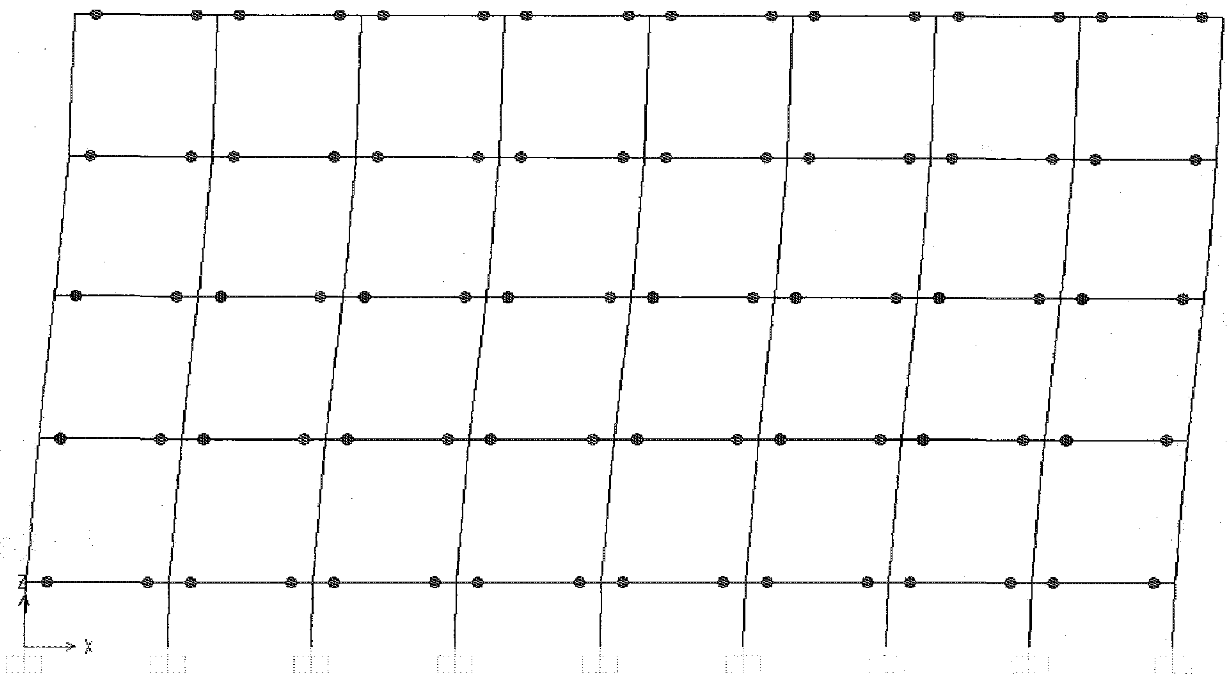

Fig. 4.18 Yield Pattern at Ultimate Point for 4-Storey Building in Longitudinal Direction Designed as SMRF with Strong Column-Weak Beam

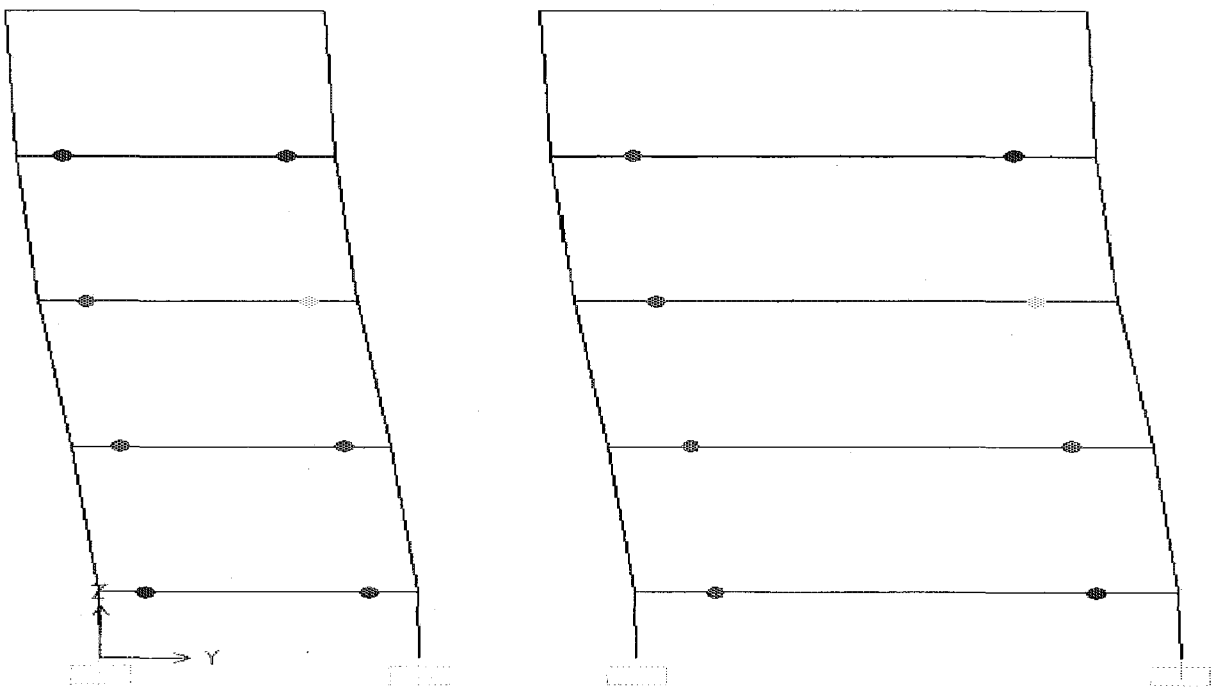

Fig. 4.19 Yield Pattern at Ultimate Point for 4-Storey Building in Transverse Direction Designed as SMRF with Strong Column-Weak Beam

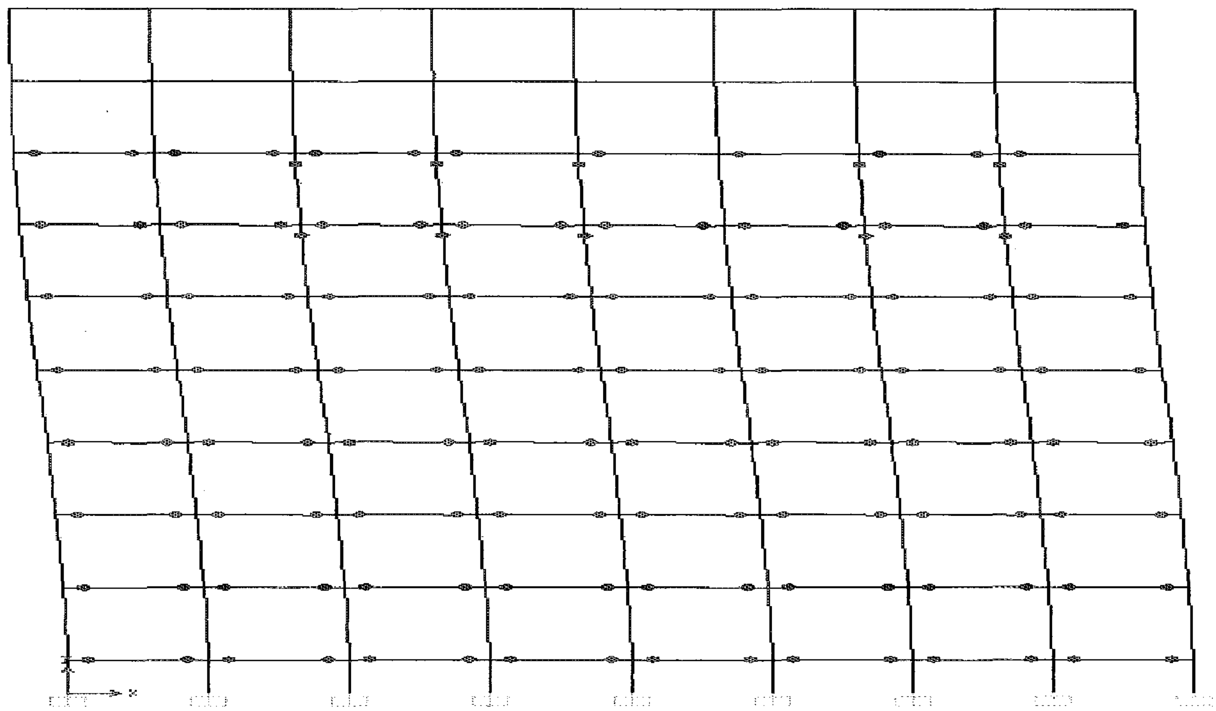

Fig. 4.20 Yield Pattern at Ultimate Point for 9-Storey Building Designed for Gravity in Longitudinal Direction

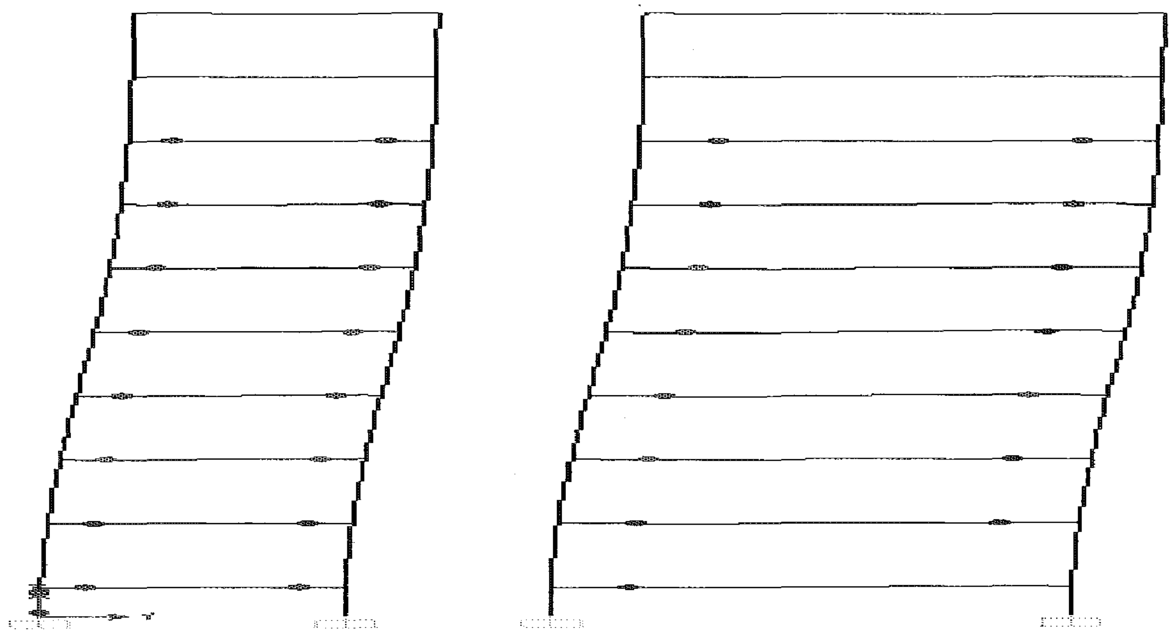

Fig. 4.21 Yield Pattern at Ultimate Point for 9-Storey Building Designed for Gravity in Transverse Direction

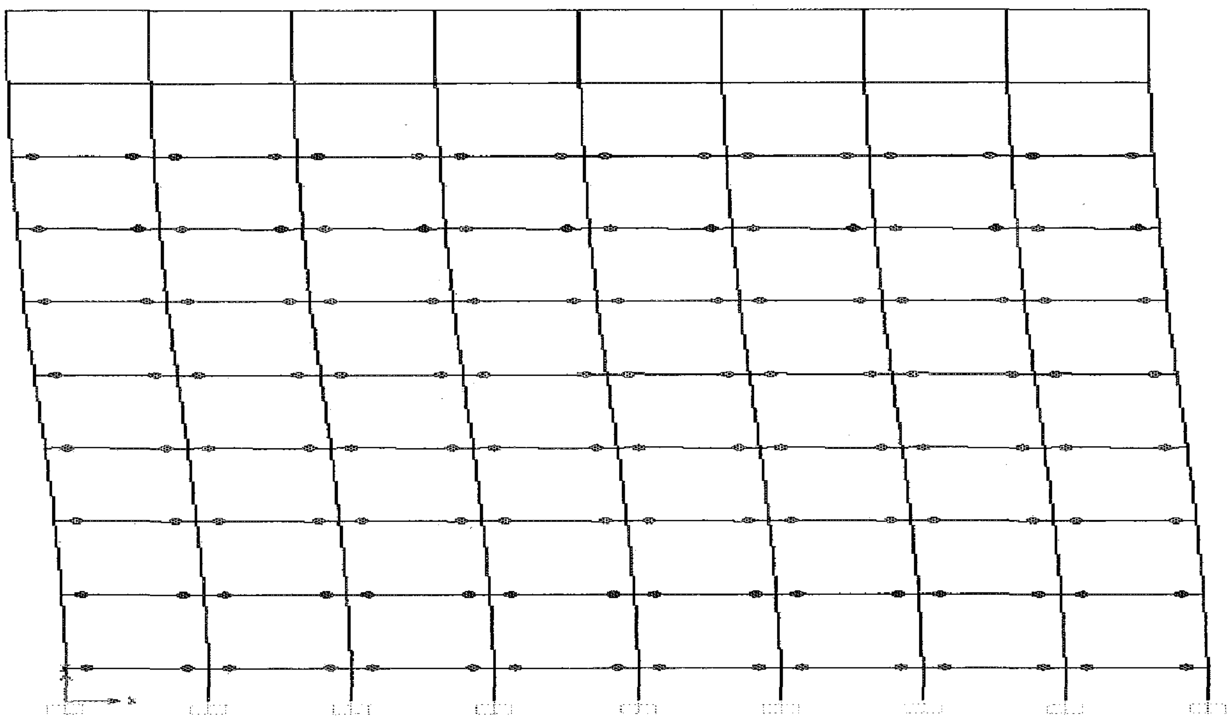

Fig. 4.22 Yield Pattern at Ultimate Point for 9-Storey Building Designed as OMRF in Longitudinal Direction

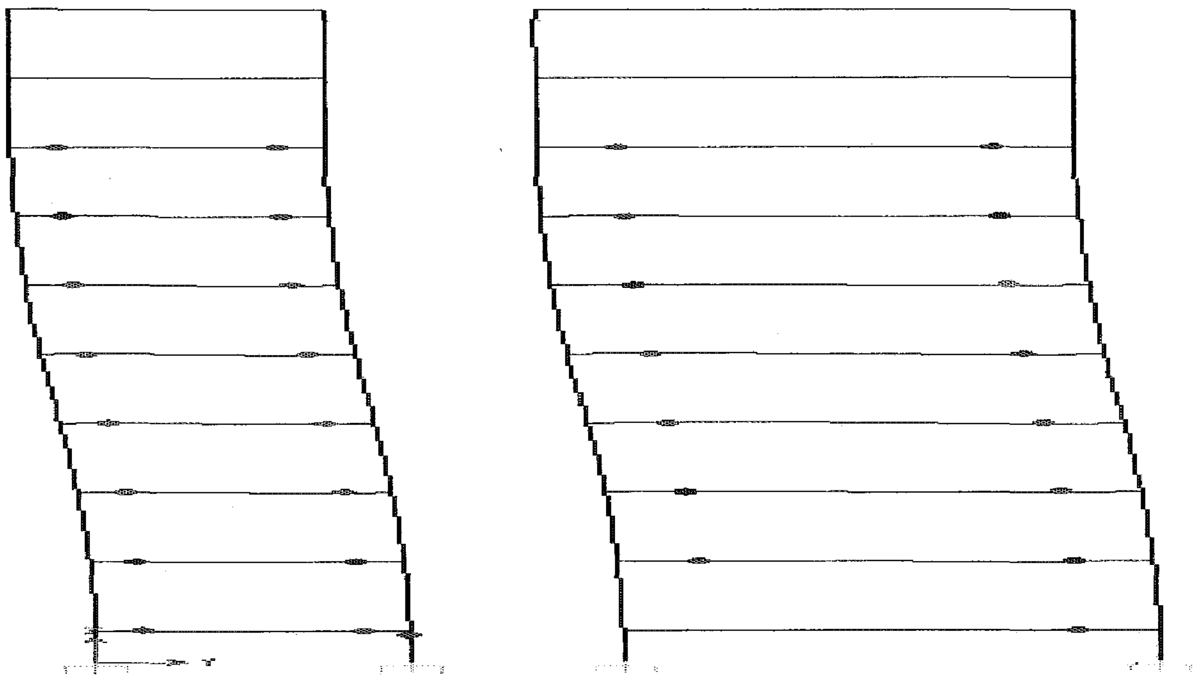

Fig. 4.23 Yield Pattern at Ultimate Point for 9-Storey Building Designed as OMRF in Transverse Direction

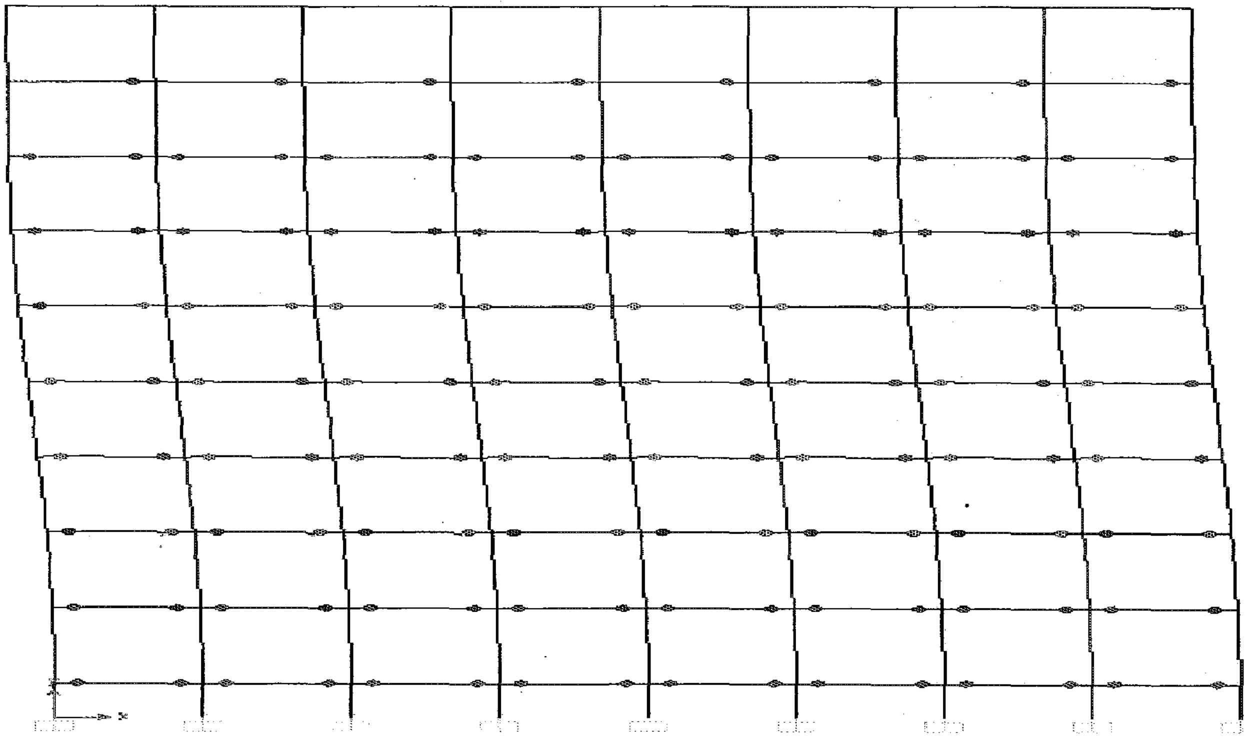

Fig. 4.24 Yield Pattern at Ultimate Point for 9-Storey Building in Longitudinal Direction Designed as SMRF with Strong Column-Weak Beam

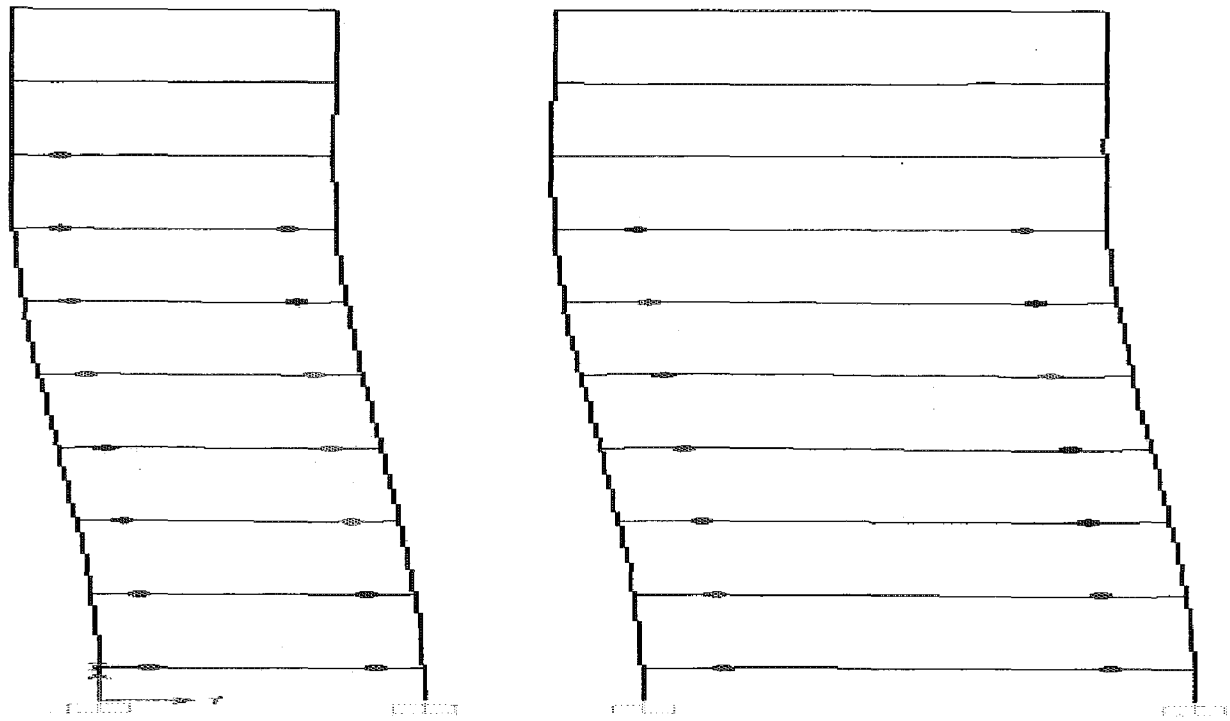

Fig. 4.25 Yield Pattern at Ultimate Point for 9-Storey Building in Transverse Direction Designed as SMRF with Strong Column-Weak Beam

4.6 SEISMIC CAPACITY PARAMETERS

As discussed in Chapter 2, the codal method of design, considers the effect of hysteretic damping, indirectly in the form of Response Reduction Factor (*R*). Actually, *R* specified in the codes represents the effect of ductility and overstrength. The relative role of these two parameters can be understood with reference to Fig. 4.26. It shows the Capacity (Pushover) Curve obtained from Nonlinear Static Analysis of the building, converted to ADRS format (Capacity Spectrum), and idealized as bilinear curve. It allows direct graphical comparison of the Capacity of the structure with the Demand imposed on it due to the expected ground shaking.

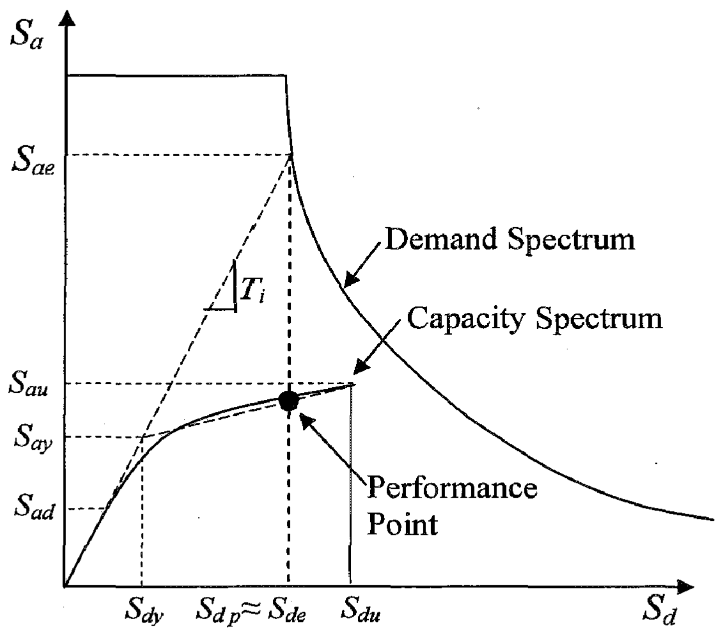

Fig. 4.26 Demand and Capacity of a typical structure represented in Acceleration-Displacement Response Spectra (ADRS) format

Capacity Spectrum can be characterized by two control points, Yield Point and Ultimate Points. Design Spectral Acceleration (S_{ad}) represents the nominal (design) strength required by the seismic code. Actually the structure is designed for this nominal strength with partial factors of safety on load combinations and material strengths. This results in overstrength and the structure actually yields at a much higher base shear represented by the Yield Spectral Acceleration (S_{ay}). In a bilinear representation, the yield point corresponds to the lateral action at which a sizeable number of members yield and beyond which the response of the structure become highly nonlinear. The Ultimate Point (S_{du}, S_{au}) represents the ultimate strength and deformation capacity of the structure. Elastic Design Strength (S_{ae}) corresponds to the hypothetical structure designed to remain elastic during the earthquake and having time period same as that of the real structure. It represents the demand on a elastic structure and is given by (generally 5%) damped elastic design response spectrum. The performance Point, representing the expected displacement of the structure is the point of intersection of the Capacity Spectrum with the Demand Spectrum, duly reduced for the effect of hysteretic damping exhibited by the structure at Performance Point. Equal Displacement Principle suggests that the displacement at Performance Point will be approximately equal to the elastic displacement. Overstrength can be defined in two ways: (i) Yield Overstrength (γ) is defined as the ratio of Yield Spectral Acceleration to the Design Spectral Acceleration (S_{ay}/S_{ad}), and Ultimate Overstrength (λ) gives the ratio of Ultimate Spectral Acceleration to the Design Spectral Acceleration (S_{au}/S_{ad}). The Ductility Demand (S_{dp}/S_{dy}) relates the Performance displacement to the Yield displacement and Ductility Capacity (S_{du}/S_{dy}) is the available ductility in the structure. The Response reduction factor, as per code, is defined as

$$R = \frac{S_{ae}}{S_{ad}} = \frac{S_{ae}}{S_{ay}} \cdot \frac{S_{ay}}{S_{ad}} = R_{eff} \cdot \gamma \tag{4.1}$$

Where, R_{eff} is the Effective Reduction Factor representing the ratio of Elastic Demand Strength to the Yield Strength. It governs the Ductility Demand on the structure. According to Equal Displacement Principle the Ductility Demand (μ) is approximately Equal to the R_{eff}, for 'Long Period' structures, while for 'Short Period' structures it is governed by Equal Energy Principle and is approximately equal to $(R_{eff}^2+1)/2$. The ductility and overstrength parameters as discussed in this section are tabulated in Tables 4.2 and 4.3 for MCE hazard level.

Table 4.2 Ductility and Overstrength Parameters for 4-Storey Building

Design Level	Longitudinal Direction					Transverse Direction				
	Overstrength		Ductility		R_{eff}	Overstrength		Ductility		R_{eff}
	γ	λ	Capacity	Demand		γ	λ	Capacity	Demand	
Gravity Designed	-	-	3.70	2.44	2.70	-	-	1.74	1.68	1.93
OMRF, Uncapped Period	2.42	2.76	3.99	2.21	2.39	2.93	4.17	1.84	1.65	1.96
OMRF, Capped Period (No Drift Control)	1.96	2.11	2.08	0.80	0.86	1.87	1.98	1.44	0.69	0.79
OMRF, Capped Period (Drift Controlled)	2.15	2.36	2.67	0.89	1.06	2.25	2.53	2.09	0.90	1.07
SMRF, Uncapped Period	3.57	4.20	5.27	2.45	2.70	5.73	7.30	1.99	1.45	1.67
SMRF, Capped Period (Capacity Design)	2.14	2.17	3.52	1.21	1.31	2.14	2.48	2.05	0.97	1.15

Table 4.3 Ductility and Overstrength Parameters for 9-Storey Building

Design Level	Longitudinal Direction					Transverse Direction				
	Overstrength		Ductility		R_{eff}	Overstrength		Ductility		R_{eff}
	γ	λ	Capacity	Demand		γ	λ	Capacity	Demand	
Gravity Designed	-	-	2.81	3.53	4.25	-	-	1.83	1.69	2.03
OMRF, Uncapped Period	1.98	1.93	3.05	2.60	2.80	3.57	3.83	1.82	1.65	1.53
OMRF, Capped Period (No Drift Control)	1.60	1.58	1.83	0.75	0.91	1.75	1.88	1.43	0.73	0.76
OMRF, Capped Period (Drift Controlled)	1.95	2.07	3.31	1.10	1.24	1.84	2.03	1.99	0.84	1.05
SMRF, Uncapped Period	2.76	2.37	3.71	2.74	3.30	4.51	5.77	1.90	1.48	2.02
SMRF, Capped Period (No Drift Control)	1.67	1.64	2.58	1.18	1.47	2.14	2.13	1.78	0.82	1.19
SMRF, Capped Period (Capacity Design)	1.68	1.72	3.14	1.22	1.57	2.23	2.39	2.62	1.11	1.25

Bilinear Capacity curves (V_B vs. D) are further converted to Capacity Spectrum (S_a vs. S_d) using FEMA-440 transformation procedure. Table 4.4 and Table 4.5 describe the pushover and capacity curve parameters of the medium and high rise RC frame buildings for different design levels.

Table 4.4 Capacity Curve and Capacity Spectrum Parameters for 4- Storey Building

		CAPACITY CURVE					**CAPACITY SPECTRUM**			
	Design Level	**Interstorey Drift**	**Yield Capacity Point**		**Ultimate Capacity Point**		**Yield Capacity Point**		**Ultimate Capacity Point**	
		D_d/H_{tot} (%)	**D_y/H_{tot}**	**V_y/W**	**D_u/H_{tot}**	**V_u/W**	**S_{dy} (cm)**	**S_{ay}(g)**	**S_{du} (cm)**	**S_{au} (g)**
	Gravity Designed	-	0.004	0.044	0.014	0.057	4.747	0.051	17.572	0.066
	OMRF, Uncapped Period	0.148	0.004	0.050	0.017	0.057	5.079	0.058	20.252	0.066
	OMRF, Capped Period	0.509	0.012	0.139	0.024	0.150	14.137	0.160	29.396	0.173
	OMRF, Capped Period (Drift Controlled)	0.320	0.007	0.153	0.020	0.168	9.417	0.179	25.186	0.197
	SMRF, Uncapped Period	0.090	0.004	0.044	0.020	0.052	4.662	0.051	24.560	0.060
	SMRF, Capped Period (Capacity Design)	0.305	0.008	0.091	0.027	0.093	9.327	0.105	32.812	0.107
Transverse Direction	Gravity Designed	-	0.006	0.054	0.011	0.073	7.65	0.062	13.312	0.084
	OMRF, Uncapped Period	0.173	0.006	0.053	0.012	0.076	7.73	0.062	14.236	0.087
	OMRF, Capped Period	0.674	0.015	0.133	0.022	0.141	18.57	0.15	26.79	0.16
	OMRF, Capped Period (Drift Controlled)	0.338	0.007	0.160	0.015	0.180	8.963	0.187	18.718	0.211
	SMRF, Uncapped Period	0.104	0.007	0.063	0.014	0.080	8.825	0.072	17.567	0.092
	SMRF, Capped Period (Capacity Design)	0.40	0.011	0.091	0.022	0.106	12.992	0.105	26.650	0.122

It is clear from the tables that for OMRF and SMRF frames, interstorey drift plays the major role in their design. Ultimate spectral displacement of SMRF is quite higher than OMRF which is desirable to facilitate large plastic deformation on the event of earthquake. The difference in capacity of gravity designed frames with code designed buildings decreases with increasing height. Gravity designed buildings will have worst performance from the earthquake point of view. Engineering properties of capacity curve such as Overstrength and ductility parameters of medium and high rise model buildings are tabulated in Tables 4.2 and 4.3, respectively.

Table 4.5 Capacity Curve and Capacity Spectrum Parameters for 8- Storey Building

	Design Level	CAPACITY CURVE					CAPACITY SPECTRUM			
		Interstorey Drift	Yield Capacity Point		Ultimate Capacity Point		Yield Capacity Point		Ultimate Capacity Point	
		D_d/H_{tot} (%)	D_y/H_{tot}	V_y/W	D_u/H_{tot}	V_u/W	S_{dy} (cm)	S_{ay} (g)	S_{du} (cm)	S_{au} (g)
Longitudinal Direction	Gravity Designed	-	0.003	0.012	0.009	0.012	7.818	0.015	21.962	0.015
	OMRF, Uncapped Period	0.163	0.004	0.019	0.013	0.018	10.538	0.023	32.149	0.022
	OMRF, Capped Period	0.570	0.013	0.064	0.024	0.064	33.176	0.078	60.589	0.077
	OMRF, Capped Period (Drift Controlled)	0.225	0.005	0.079	0.017	0.084	13.793	0.095	45.669	0.101
	SMRF, Uncapped Period	0.098	0.004	0.016	0.015	0.013	10.098	0.019	37.462	0.016
	SMRF, Capped Period (Capacity Design)	0.445	0.008	0.040	0.021	0.040	20.813	0.049	53.723	0.048
Transverse Direction	Gravity Designed	-	0.008	0.024	0.014	0.032	19.301	0.030	35.225	0.039
	OMRF, Uncapped Period	0.248	0.008	0.032	0.014	0.034	19.545	0.039	35.589	0.042
	OMRF, Capped Period	0.639	0.015	0.071	0.021	0.076	37.374	0.086	53.491	0.092
	OMRF, Capped Period (Drift Controlled)	0.338	0.009	0.074	0.018	0.082	23.209	0.090	46.107	0.099
	SMRF, Uncapped Period	0.108	0.008	0.024	0.014	0.031	19.301	0.030	36.729	0.038
	SMRF, Capped Period (Capacity Design)	0.543	0.011	0.052	0.020	0.052	28.651	0.063	51.045	0.062

4.7 DISCUSSION

It can be observed from Pushover curves that earthquake forces govern the design in the present case (Seismic Zone – IV) for 4 storey, as well as, for 9 storey building. Consideration of earthquake effects, as per IS:1893, in the design increases the strength and ductility capacities of the building. However, the relative increase depends strongly on the building height, design period of vibration, and the span of the beams in the direction under consideration. While, in the transverse (having longer span of beams) direction of the 4 storey building, the increase in capacity is about 20%, in the longitudinal direction of the 9 storey building it is about 300%. It is also interesting to note that in Seismic Zone-IV, the building designed without any consideration for earthquake forces, gives Collapse Prevention (CP) performance level, even for MCE (except for 9 storey building, with earthquake acting in longitudinal direction). This means that even if the building is designed and constructed properly for the gravity loads, alone, as per the relevant IS codes (IS:875 and IS:456) it has sufficient overstrength and ductility to survive, without collapse, the DBE (and in most of the cases even MCE) level of ground shaking specified by IS:1893 for Seismic Zone IV. Of course, for the buildings designed for seismic effects, the performance is much better and it can be observed from the Figs. that the estimated performance is better than Immediate Occupancy (IO) in all the cases considered. Buildings designed in accordance with the present seismic code requirements for DBE, show IO level Seismic Performance for MCE, in most of the cases. SMRF buildings have enhanced ductility but reduced strength, as compared to OMRF buildings. The performance of SMRF buildings has been observed to be only marginally better than OMRF buildings.

SEISMIC VULNERABILITY OF RC FRAME BUILDINGS

5.1 GENERAL

The Seismic Vulnerability of a structure is described as the susceptibility to damage by ground shaking of a given intensity. The aim of Vulnerability Assessment is to predict cost of repair, as a ratio of the cost of replacement, which eventually gives the loss estimation, for the seismic hazards at the sites under consideration. In vulnerability assessment procedure, a parameter is selected to characterize the ground motion and it is related with damage to buildings. Traditionally, Seismic Intensity and Peak Ground Acceleration (PGA) have been used to represent vulnerability, while nowadays the Seismic Vulnerability of buildings is correlated with response spectra obtained from the ground motion. For representing the vulnerability, Fragility Curves and Damage Potential Matrices (DPM) are the most commonly used formats. Fragility curves express the data in a graphical format while DPM express it numerically. Both methods describe the conditional probability of exceeding different levels of damage at given levels of ground motion severity. In the present study, fragility curves and Damage Potential Matrices (DPM) are adopted to represent vulnerability.

5.2 VULNERABILITY ASSESSMENT METHODOLOGIES

The available vulnerability assessment procedures are classified based on the degree of sophistication involved, quality of data in hand and the purpose of the assessment. There are basically two categories of vulnerability assessment - empirical and analytical methods. There is also a hybrid method, in which both empirical and analytical methods are used. *Fig 5.1* shows the schematic classification of the various methods for vulnerability assessment.

5.2.1 Empirical Methods

Empirical vulnerability assessment methods are based on statistics of past earthquake damage and can be summarized and represented via Damage Probability Matrices (DPM) (Withman 1973), or vulnerability/ fragility curves (Rossetto and Elnashai, 2003).

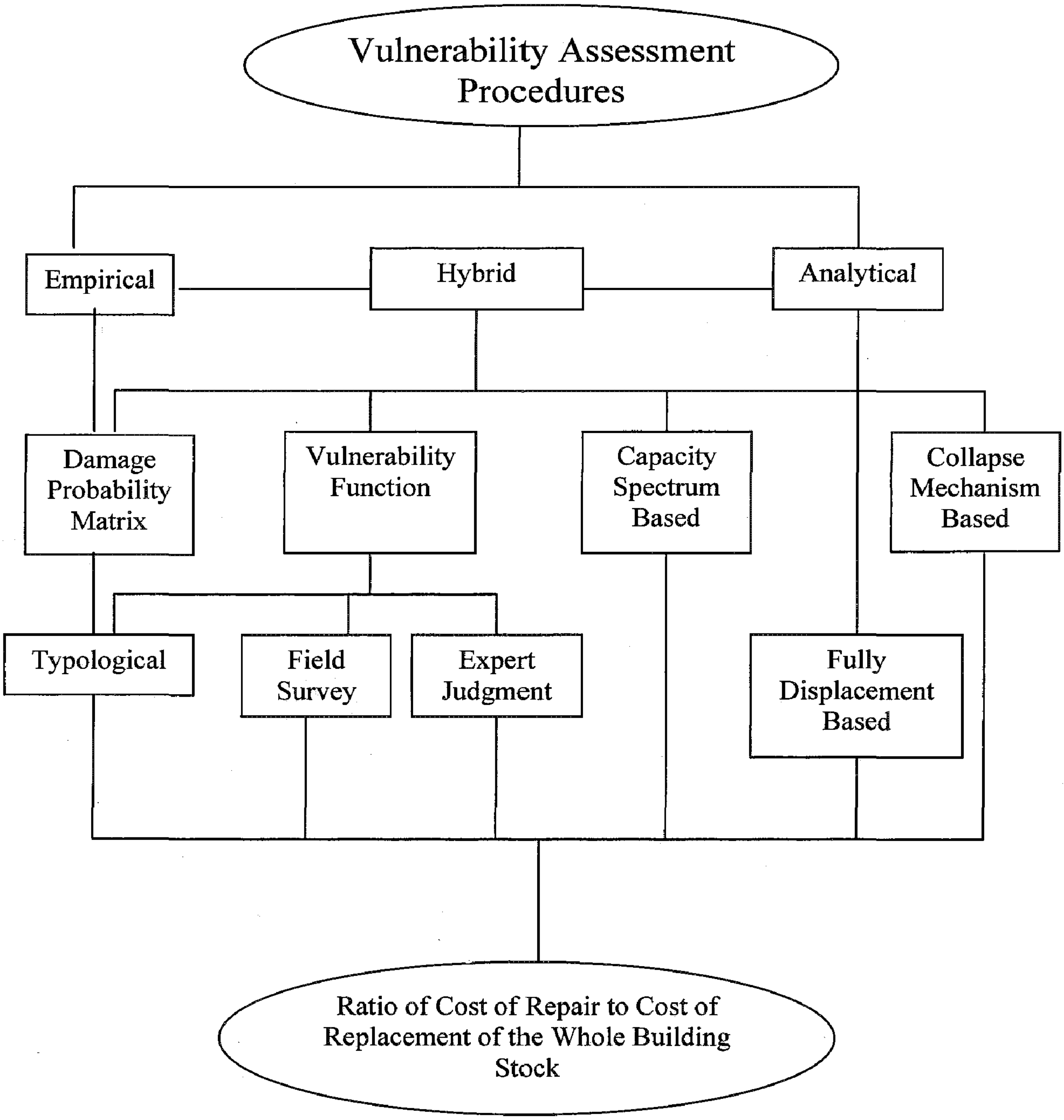

Fig. 5.1 Vulnerability Assessment Procedures (Calvi et al., 2006)

5.2.2 Analytical Methods

Mechanical or analytical vulnerability models for a wide regional scale analysis for classes of buildings can be defined on the basis of either traditional Force-Based procedure (e.g., capacity Spectrum Method implemented in HAZUS, 1999 or RISK_UE, 2004) or according to more recent proposals, Displacement–Based Designed approaches (Calvi et al. 2005). According to Force-Based procedures, the building performance is identified, within a ADRS (Acceleration-Displacement Response Spectra) domain, by the intersection point between the Capacity Curve of an equivalent non linear SDOF system and the earthquake Demand Curve, duly reduced to account for the inelastic behavior and energy dissipation capacity of the system. While, according to Displacement–Based approaches, the periods associated to the boundary of different limits states can be evaluated by the intersection between capacity curves, represented in terms of period-displacement relationship, and the displacement spectrum demand curves, scaled by equivalent viscous damping factors. Other proposals for mechanical-based methods are based on the evaluation of collapse multipliers associated to alternative collapse mechanisms or on the derivation of vulnerability or fragility curves from the results of extensive numerical analyses.

5.2.3 Hybrid Methods

Due to the inherent difficulty to retrieve reliable and exhaustive observed damage data, referred to all defined building typologies, earthquake intensities and soil conditions, "hybrid" methodologies can be implemented, relying on the combination of the available empirical/statistical data with the results of either numerical analyses (Kappos et al. 1995), Neural Network systems and Fuzzy Set Theory or more directly, using expert judgments. Expert-based vulnerability methods apply human judgment to completely replace the processing of observed data, leading to experts-defined DPM (e.g. ATC-13, 1987) or score assignment procedures (e.g. ATC-21 1988, FEMA-154). The main difficulty of hybrid methods is to calibrate analytical result based on the observed data, since the two vulnerability curves incorporate different sources of uncertainty and therefore are not directly comparable.

5.3 HAZUS METHODOLOGY

HAZUS methodology was developed for FEMA by National Institute of Building Science (NIBS) to reduce seismic hazard in United States and has been used, in some form or other, all over the world, for loss assessment of urban areas. The methodology deals with nearly all aspects of built environment, and with a wide range of different types of losses. This methodology has been used to develop vulnerability/fragility curves for buildings and other facilities to estimate losses due to ground shaking which is further used to define likely losses for a range of different building type in United States. There are total 36 classes of buildings. The main modules involved in the methodology are given in the *Fig. 5.2.*

The vulnerability assessment part comes under the direct physical damage module and is based on the Capacity Spectrum Method of ATC-40. In this method, ground motion is represented by an Acceralation-Diaplacement spectrum and the horizontal displacement of the structure under increasing lateral load is represented by Capacity Spectrum. The reduction of spectrum is applied to account for the hysteretic damping that takes place during the inelastic response of the structure. Hysteretic damping is obtained from the area enclosed by the hysteretic loop at peak response displacement and acceleration. Finally, the performance point of the building under consideration is obtained from the intersection of the demand and the capacity curve for a particular ground motion. *Fig 5.3* shows the process of damage estimation from ground motion given in HAZUS methodology. The capacity spectrum has been developed for each building class using model buildings.

The Performance Points obtained for different building classes, are the displacement input for the vulnerability/fragility curves to give the probability of being in a particular damage level (none, slight, moderate, extensive and complete). The vulnerability curves are lognormal curves with a logarithmic standard deviation, β_{sds}, calculated as

$$\beta_{sds} = \sqrt{(CONV[\beta_C \beta_D \bar{S}_{d,sds}])^2 + (\beta_{M,sds})^2]} \qquad (5.1)$$

where,

β_C = Variability in the capacity properties of the model building type,

β_D = Uncertainty in the response due to spatial variability of the ground motion demand,

$\bar{S}_{d,sds}$ = Median spectral displacement for damage state *ds*, and

$\beta_{M,sds}$ = Uncertainty in the damage state threshold.

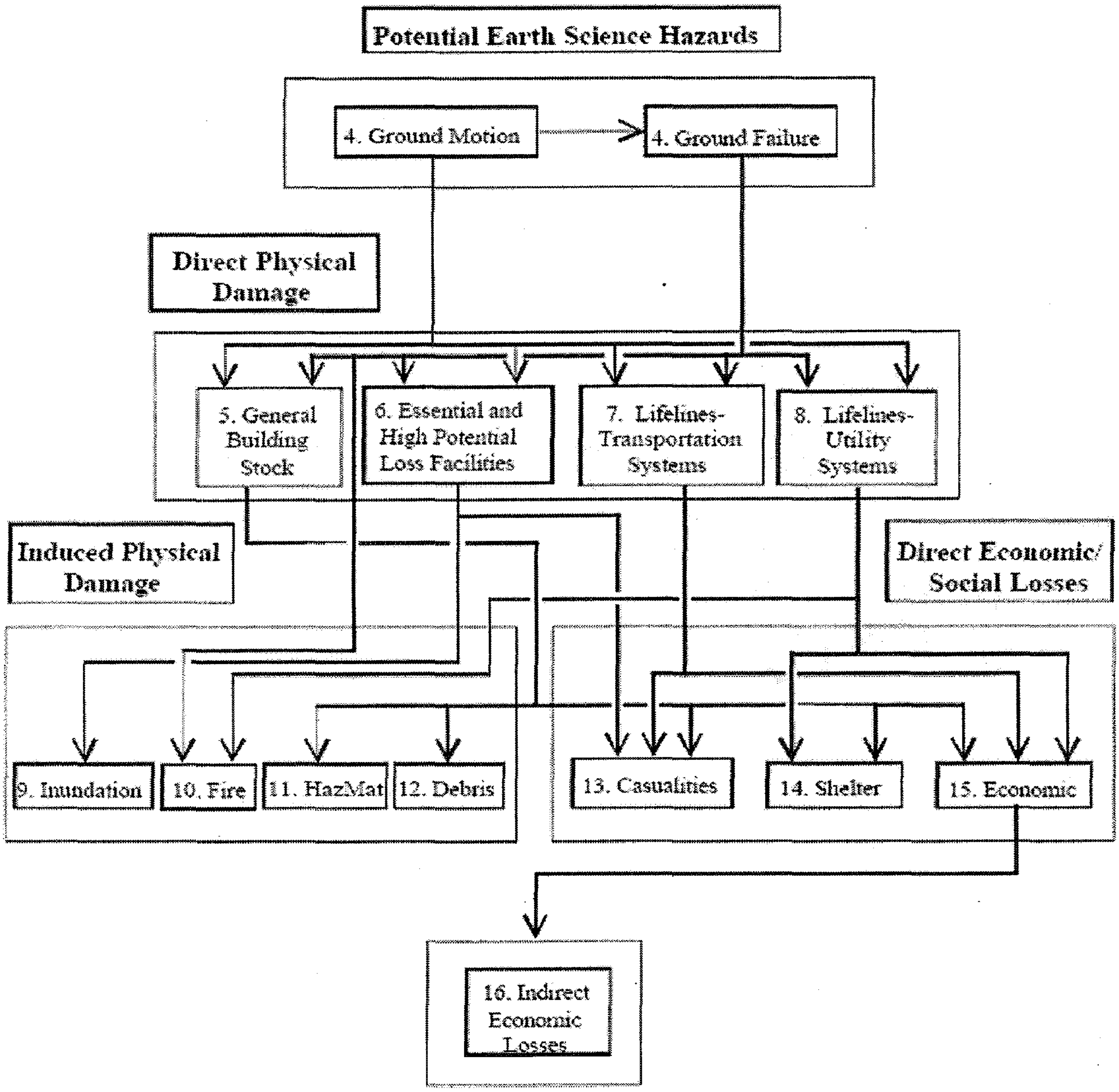

Figure 5.2 Flow Chart for Earthquake Loss Estimation as Per HAZUS Methodology (HAZUS-MH)

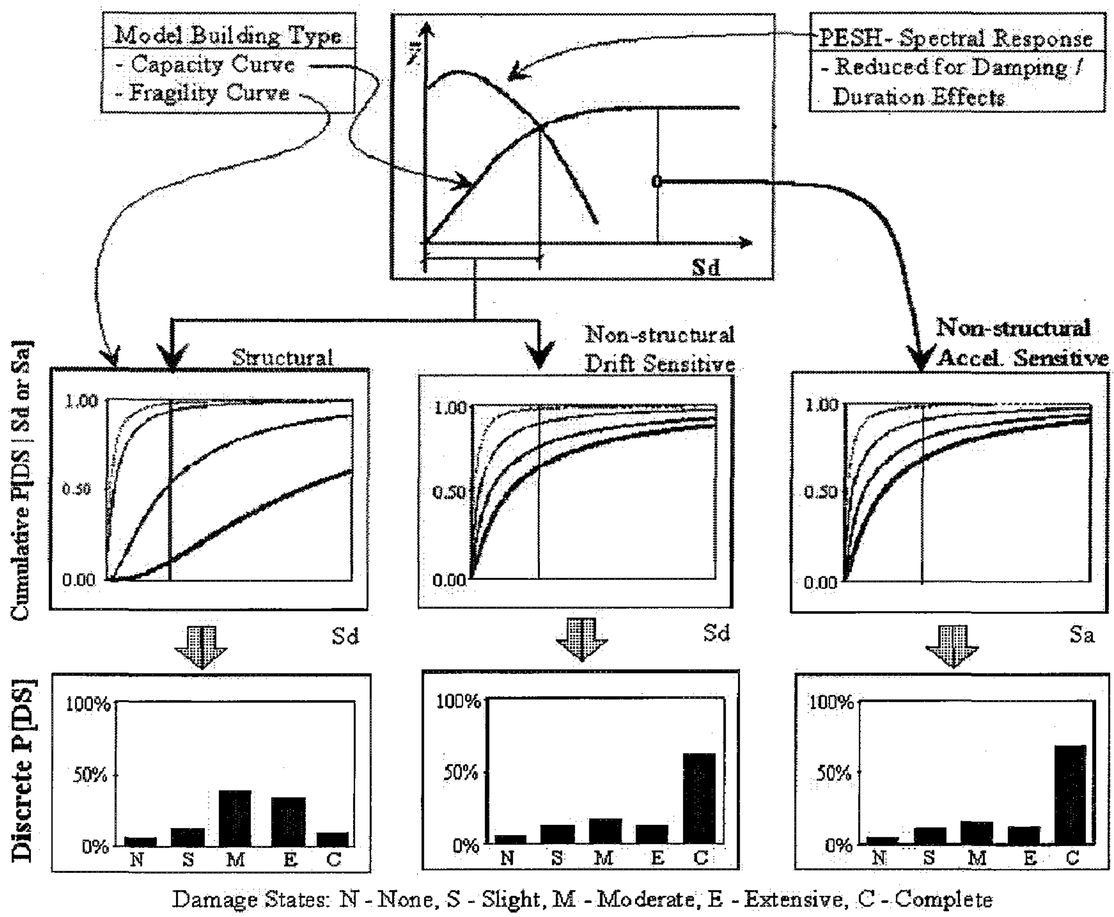

Figure 5.3 Building Damage Estimation Process as per HAZUS Methodology (HAZUS-MH)

The main disadvantage of HAZUS method is that the capacity curves and vulnerability curves published in the HAZUS manual has been derived for US buildings and therefore application of this method to other part of world requires additional research.

5.4 DEFINITION OF DAMAGE STATES

Although damage to the structure is a continuous function of earthquake demand, but it is defined by the discrete damage states such as, No Damage, Slight Damage, Moderate Damage, Heavy Damage, and Collapse, as subsets of complete damage of the structure, since it is not required to have continuous scale for understanding building's physical condition. Vulnerability/ Fragility curves are represented with respect to spectral displacement, as interstorey drift is a very good indicator of damage. When appropriate Capacity Curves are available, Fragility Curves are derived by defining damage states in terms of structure displacements (typically roof drift) and transforming (used in HAZUS) them into spectral displacement i.e., displacement of equivalent single degree of freedom system. For the present study, four damage states, Slight Damage (Gr1), Substantial to Heavy Damage (Gr3), Very Heavy Damage (Gr4) are considered to represent the physical condition of buildings from engineering requirements point of view. Table 5.1 depicts the engineering limit states for the damage states, as defined by Kappos et.al, 2006.

Table 5.1 Damage State Definition (Kappos et al, 2006)

Damage Grade	Damage State	Spectral Displacement
Gr0	None	$0.7S_{dy} < S_d$
Gr1	Slight Damage	$0.7S_{dy} < S_d < S_{dy}$
Gr2	Moderate Damage	$S_{dy} \leq S_d < 2S_{dy}$
Gr3	Substantial to Heavy Damage	$2S_{dy} < S_d < 0.7S_{du}$
Gr4	Very Heavy Damage	$0.7S_{du} < S_d < S_{du}$
Gr5	Collapse	$S_d > S_{du}$

5.5 CONSIDERATION OF UNCERTAINTY

The total variability in structural damage is contributed by the three sources, β_C, β_D, $\beta_{M,sds}$, combined using a complex convolution process as given in equation (5.1). Alternatively, the uncertainties can be handled using the proper values given for medium and high-rise buildings in HAZUS-MH Technical manual. Tables 5.2-5.3 show the different variabilities considered for the study.

Table 5.2 Uncertainty Considerations for 4 -Storey Buildings (HAZUS-MH)

Design Levels	**Post-yield degradation (κ)**	**Damage Variability ($\beta_{T,ds}$)**	**Capacity Curve Variability (β_c)**	**Total Variability (β_{sds})**
Gravity Designed	Major Degradation (0.5)	Moderate (0.4)	Moderate (0.3)	0.85
OMRF, Uncapped Period				
OMRF, Capped Period				
OMRF, Capped Period (Drift Controlled)				
SMRF, Uncapped Period	Minor Degradation (0.9)			0.75
SMRF, Capped Period				
SMRF, Capped Period (Capacity Design)				
SMRF, Equalized Drift				

Table 5.3 Uncertainty Considerations for 9- Storey Buildings (HAZUS-MH)

Design Levels	**Post-yield degradation (κ)**	**Damage Variability ($\beta_{T,ds}$)**	**Capacity Curve Variability (β_c)**	**Total Variability (β_{sds})**
Gravity Designed	Major Degradation (0.5)	Moderate (0.4)	Moderate (0.3)	0.80
OMRF, Uncapped Period				
OMRF, Capped Period				
OMRF, Capped Period (Drift Controlled)				
SMRF, Uncapped Period	Minor Degradation (0.9)			0.70
SMRF, Capped Period				
SMRF, Capped Period (Capacity Design)				
SMRF, Equalized Drift				

5.6 FRAGILITY CURVES

HAZUS methodology has been used to develop Vulnerability/ Fragility curves for RC buildings of different design levels and heights. The Performance Point obtained from Capacity Spectrum developed for each design level, using model buildings of different building classes, is the displacement input for the Vulnerability/ Fragility curves to give the probability of building being in a particular damage level. The probability of being or exceeding a given damage state is calculated as

$$P[ds/S_d] = \Phi[\frac{1}{\beta_{ds}} \ln(\frac{S_d}{\bar{S}_{d,ds}})] \qquad (5.2)$$

Where,

$\bar{S}_{d,ds}$ = Median spectral displacement for damage state ds,

Φ= Normal cumulative distribution function, and

β_{ds} = Standard deviation of the threshold spectral displacement.

Figures 5.4 - 5.9, show the Fragility Curves of different design levels of 4-storey and 9-storey model buildings.

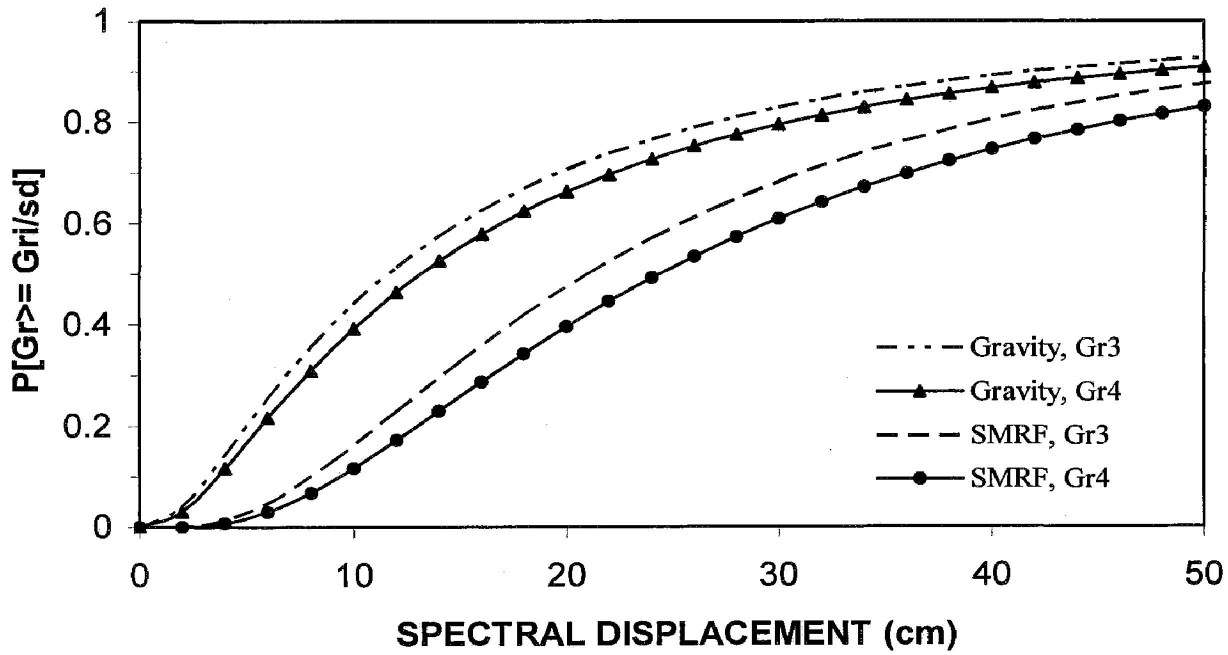

Fig. 5.4 Comparison of Vulnerability for Damage Grades Gr3 and Gr4 of 4-Storey Building Designed for Gravity Load only and as SMRF, as per Relevant IS codes

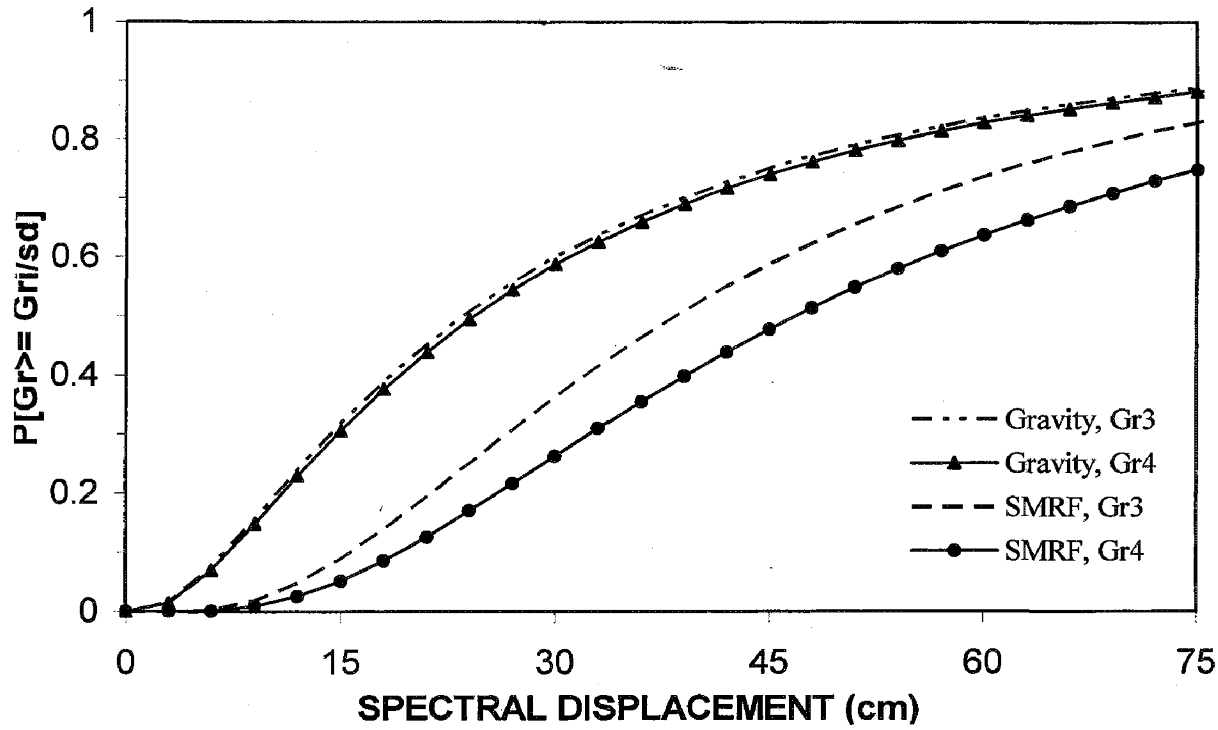

Fig 5.5 Comparison of Vulnerability for Damage Grades Gr3 and Gr4 of 9-Storey Building Designed for Gravity only and as SMRF, as per Relevant IS codes

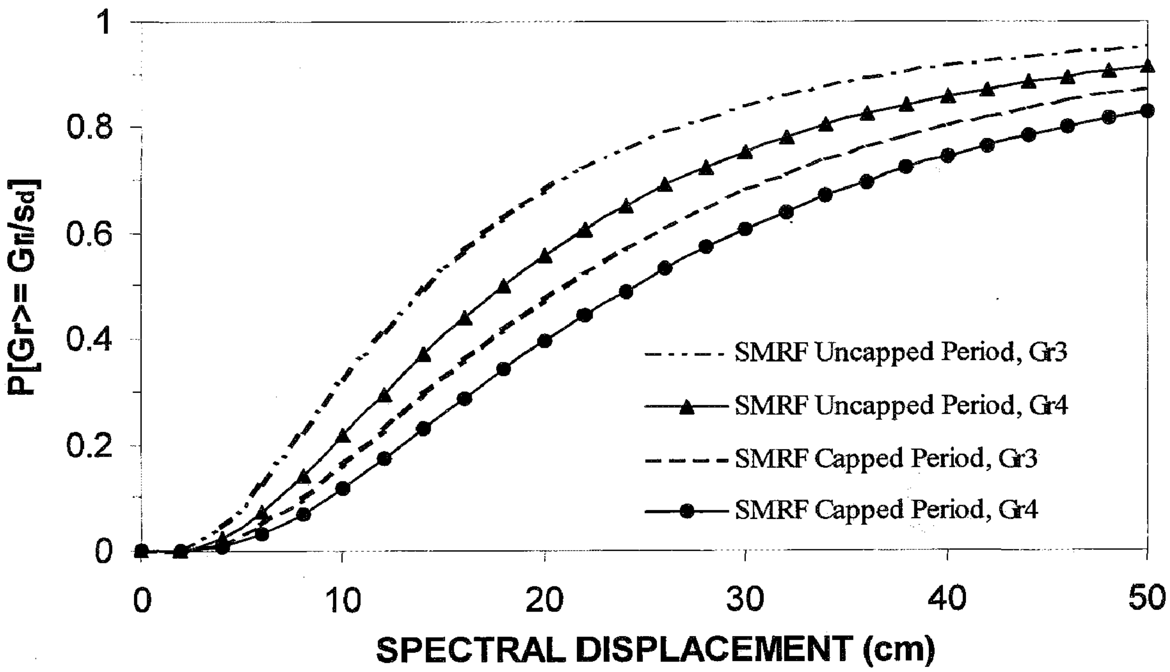

Fig 5.6 Comparison of Vulnerability for Damage Grades Gr3 and Gr4 of 4-Storey Building Designed as SMRF, with and without Period Capping

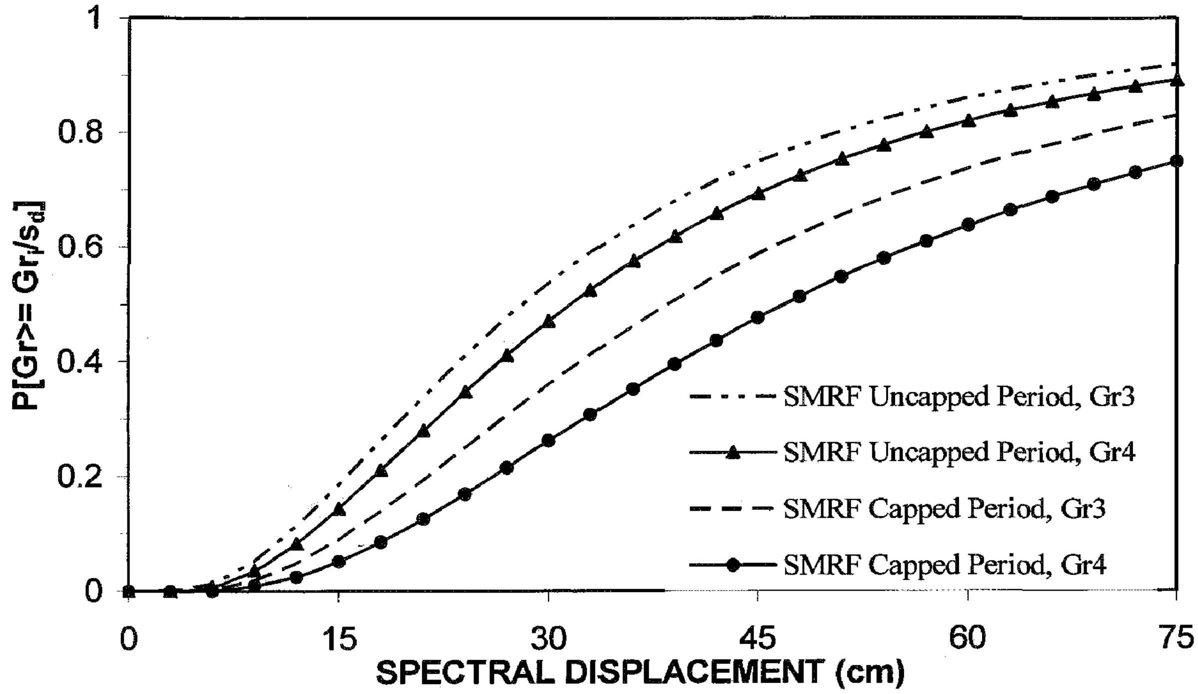

Fig 5.7 Comparison of Vulnerability for Damage Grades Gr3 and Gr4 of 9-Storey Building Designed as SMRF, with and without Period Capping

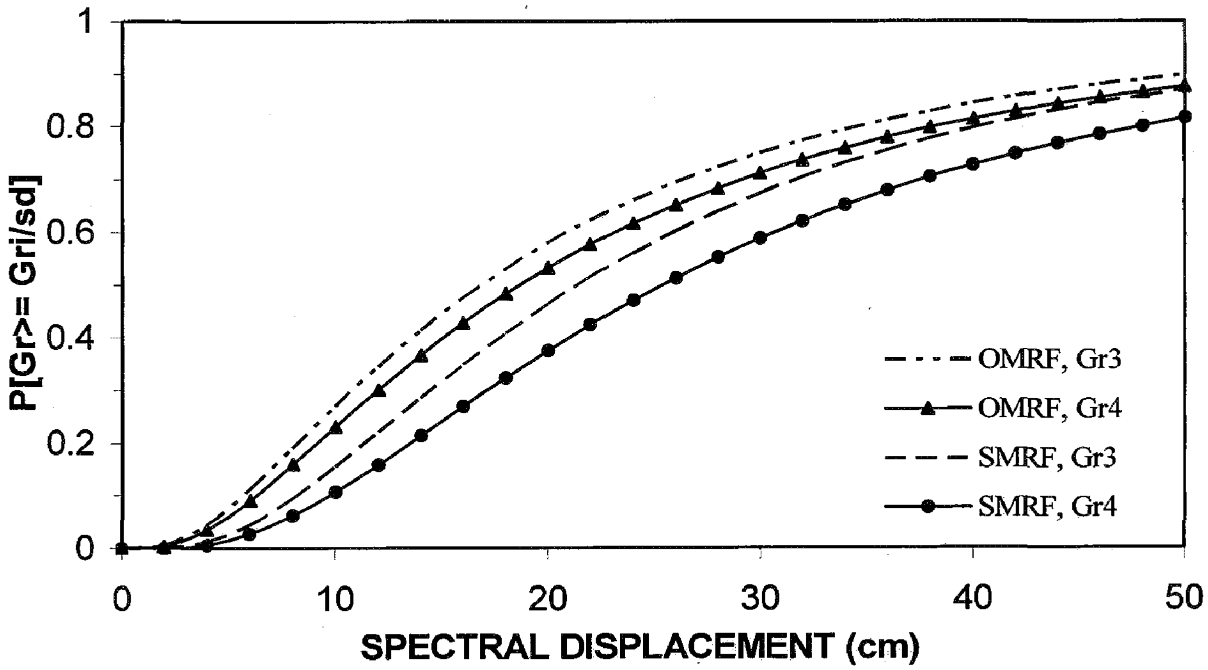

Fig 5.8 Comparison of vulnerability for Damage Grades Gr3 and Gr4 of 4-Storey Building Designed as OMRF and SMRF

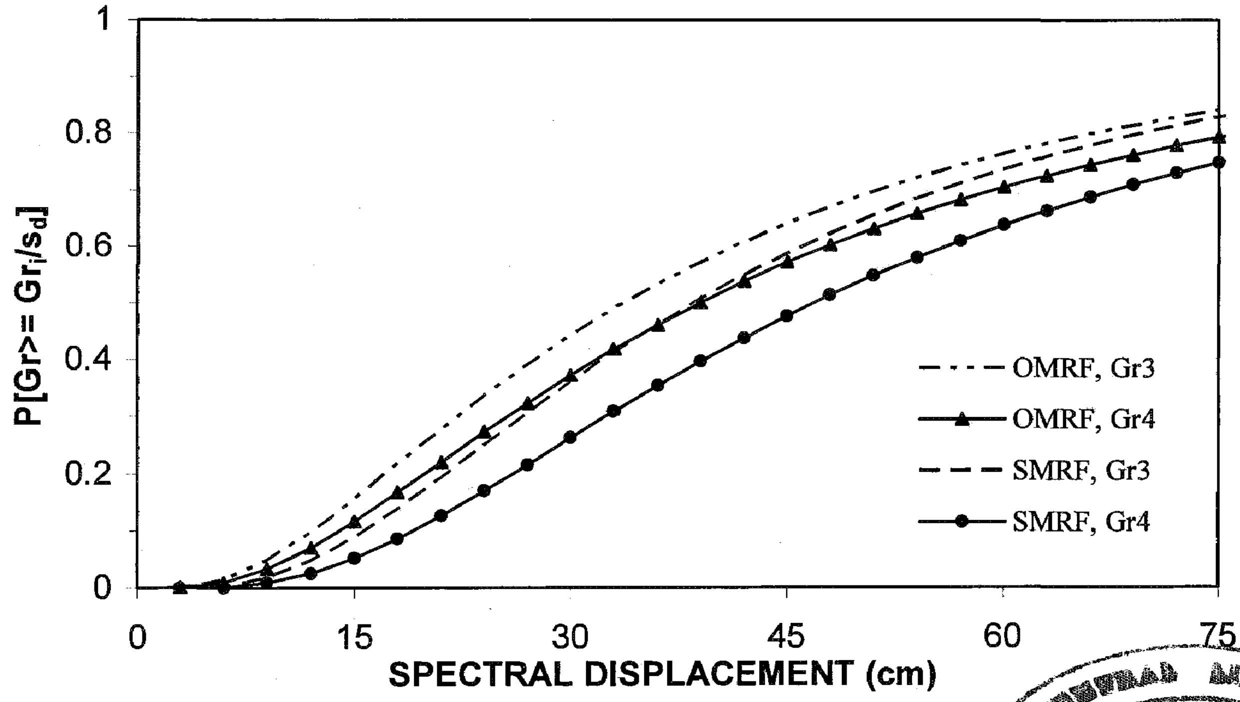

Fig 5.9 Comparison of vulnerability for Damage Grades Gr3 and Gr4 of 9-Storey Building Designed as OMRF and SMRF

5.7 DAMAGE PROBABILITY MATRICES (DPMs)

The main concept of DPM is that, a particular structural typology will have same probability of being in a damage level due to a given earthquake Intensity or Peak Ground Acceleration. Fragility curves represent vulnerability in terms of spectral displacement. But to have an idea of vulnerability of the buildings of different design level at a given Peak Ground Acceleration (PGA), DPMs are a must. Tables 5.4 and 5.5 show the vulnerability of the of 4-storey and 9-storey model buildings of different design level at design (0.12g) and higher PGA, as the PGA higher than that used for design are expected to be encountered by the building during its life time.

Table 5.4 Damage Probabilities (%) For 4-Storey RC Buildings

Design Level	Damage ≥Gr.1 for PGA (g)				Damage ≥Gr.3 for PGA (g)				Damage ≥Gr.4 for PGA (g)			
	0.12	**0.18**	**0.24**	**0.36**	**0.12**	**0.18**	**0.24**	**0.36**	**0.12**	**0.18**	**0.24**	**0.36**
Gravity Designed	52.75	70.75	81.18	91.33	19.94	35.69	48.87	67.32	8.06	30.43	43.11	61.92
OMRF, Uncapped Period	50.10	68.42	79.33	90.23	17.24	32.01	44.86	63.61	5.36	25.45	37.38	56.16
OMRF, Capped Period	18.92	34.31	47.39	65.96	3.70	9.51	16.56	31.05	2.84	7.66	13.79	27.00
SMRF, Uncapped Period	52.24	70.26	80.28	90.36	15.54	28.67	40.38	58.20	2.76	20.40	30.59	47.72
SMRF, Capacity Design	23.54	42.83	58.04	77.14	3.44	10.05	18.54	36.16	2.10	6.77	13.37	28.48
SMRF, Equalized Drift	23.98	43.39	58.60	77.57	0.12	0.65	1.79	5.95	0.51	2.12	4.99	13.46

Table 5.5 Damage Probabilities (%) for 9-Storey RC Buildings

Design Level	Damage ≥Gr.1 for PGA (g)				Damage ≥Gr.3 for PGA (g)				Damage ≥Gr.4 for PGA (g)			
	0.12	**0.18**	**0.24**	**0.36**	**0.12**	**0.18**	**0.24**	**0.36**	**0.12**	**0.18**	**0.24**	**0.36**
Gravity Designed	53.14	72.09	82.77	92.67	20.74	37.88	52.03	71.15	19.66	36.43	50.50	69.82
OMRF, Uncapped Period	47.83	67.45	79.16	90.64	16.25	31.66	45.32	65.14	14.25	28.70	41.97	61.95
OMRF, Capped Period	14.74	29.43	42.81	62.76	2.11	6.36	12.19	25.50	1.45	4.68	9.39	20.90
SMRF, Uncapped Period	49.18	68.66	80.12	91.19	15.51	30.57	44.10	63.99	11.95	25.12	37.78	57.75
SMRF, Capacity Design	19.88	39.48	55.73	76.53	1.91	6.76	13.95	30.73	0.93	3.79	8.62	19.62
SMRF, Equalized Drift	26.62	48.20	64.27	82.77	0.08	0.52	1.58	5.83	0.37	1.81	4.61	13.26

5.8 DISCUSSION

Seismic Vulnerability greatly depends on the design level and probability of damage increases with increase in building height for all grades of damages. It is evident from the DPMs that gravity load designed building pose worst damage threat although it has possessed considerable overstrength and ductility to survive against collapse at MCE.Buildings designed as SMRF as per IS:1893 and IS:13920 pose higher probability of damage in lower damage state (Gr.1) than buildings designed as OMRF as per IS:1893 for a particular PGA. In the present study, damage probability is calculated with different uncertainty for different design levels, because it is not rational to assume same uncertainty factor for all design levels.

Damage probability of higher damage grades (Gr.3 and Gr.4) of SMRF design is quite close to that of OMRF. This is because the allowable drift limits in IS:1893 have been defined at design force level, which means that the SMRF, is designed for total drift 5/3 times higher than that of OMRF. It has been shown that if limit is applied on total drift, as in case of IBC and EC8, the damage probability for SMRF is much less than that for OMRF.

EFFECT OF URM INFILLS

6.1 GENERAL

For the purpose of studying the effect of URM infills on the Seismic Performance and Vulnerability of RC frame buildings for gravity and SMRF design levels, the same 4 - storey (G+3) RC building (Fig 4.1) has been modeled with uniform infills. The thickness of the walls has been considered as 115 mm and 230 mm for interior and exterior walls, respectively. Infill panels are assumed to be solid for the purpose of stiffness and strength modeling. The density of masonry infill has been considered $20 kN/m^3$. The RC frames with infill mass have been designed as per the design requirements of IS: 456(2000) and IS: 1893- Part I (2002). Infill effect has been incorporated in the design by considering the correction factor for the increased base shear for the rigidity of the building due to presence of URM infills. A comparative study between the bare and the uniformly infilled RC frame buildings has been presented in terms of dynamic characteristics, seismic performance, yield pattern, and finally the seismic Vulnerability/Fragility.

6.2 MODELLING OF URM INFILL

Simulation of actual behavior of infilled frame is difficult and complex since it exhibits highly nonlinear and inelastic behavior due to infill-frame interaction. Unreinforced masonry infill can be modeled as micro models and macro models, respectively. Micro models are based on finite element representation of each infill panel and thus capture the behavior and its interaction with frame in a much detail manner, but they are computationally very expensive where as macro models are based on physical understanding of infill panel as a whole and is able to simulate the gross behavior of infill, but they are computationally efficient although approximate.

6.2.1 Linear Modelling of URM Infill

The linear modelling of URM infill consists of simulation of its stiffness relative to different component of the building as forces are distributed in the ratio of stiffness.

For this present study, Infill panels have been modelled as equivalent diagonal compressive strut element. Struts have been concentrically placed across the diagonals of the frame as shown in Fig. 6.1.

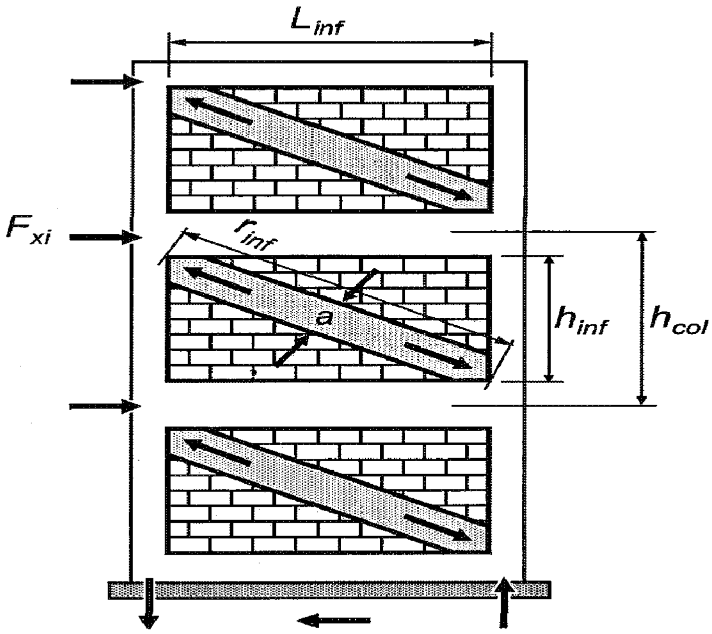

Fig. 6.1 Concentric Compression Strut Model as per FEMA-356

The thickness and modulus of elasticity of the strut is kept same as that of infill. It has zero out of plane stiffness. The elastic in-plane stiffness of a solid Unreinforced masonry infill panel prior to cracking is idealized as an equivalent diagonal compression strut of width, 'a', is defined by FEMA 356 as,

$$a = 0.175(\lambda_1 h_{col})^{-0.4} r_{\text{inf}} \tag{6.1}$$

where,

$$\lambda_1 = \left[\frac{E_{me} t_{\text{inf}} \sin 2\theta}{4 E_{fe} I_{col} h_{\text{inf}}}\right]^{\frac{1}{4}}$$

h_{col} = column height between centerlines of beams

h_{inf} = height of infill panel

E_{fe} = expected modulus of elasticity of frame material (concrete)

E_{me} = expected modulus of elasticity of infill material

I_{col} = moment of inertia of column

L_{inf} = length of infill panel

r_{inf} = diagonal length of infill panel

t_{inf} = thickness of infill panel and equivalent strut

Link elements with gap have been inserted in the struts so that struts behave as a normal bracing element (active in both compression and tension) in linear analysis. The elastic properties of link elements have been computed in such a way that it will not change the linear stiffness of the infill panel. This action of strut with link elements as gap elements in linear analysis is described in Fig. 6.2.

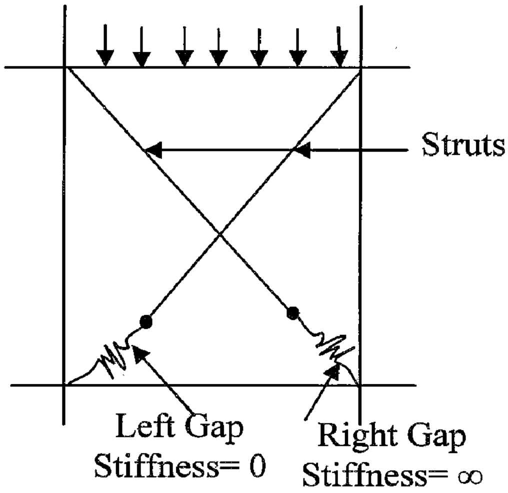

Fig. 6.2 Strut Model of infill for Linear Analysis under Gravity Load

6.2.2 Non-Linear Modelling of URM Infill

In nonlinear analysis, in addition to the stiffness, strength of infill is also simulated. The basic steps described in section 6.2.1 for linear modelling are required in nonlinear modelling, as well. In addition to these, the strength properties in different actions, including the nonlinear load-deformation curves, are also modelled for each strut members. Strength of each strut member is calculated based on the minimum strength considering all possible failure modes (compression failure, buckling and sliding shear failure of diagonal strut) of infill (Pauley, T. and Priestley, 1992). Nonlinearity has been considered in each strut element by providing axial (P) plastic hinges at mid-length of the struts. Table 7-9 of FEMA 356 was used for calculation of hinge properties. Fig. 6.3

defines the generalized force-deformation relationship of the plastic hinges used to define the performance limit states for masonry members.

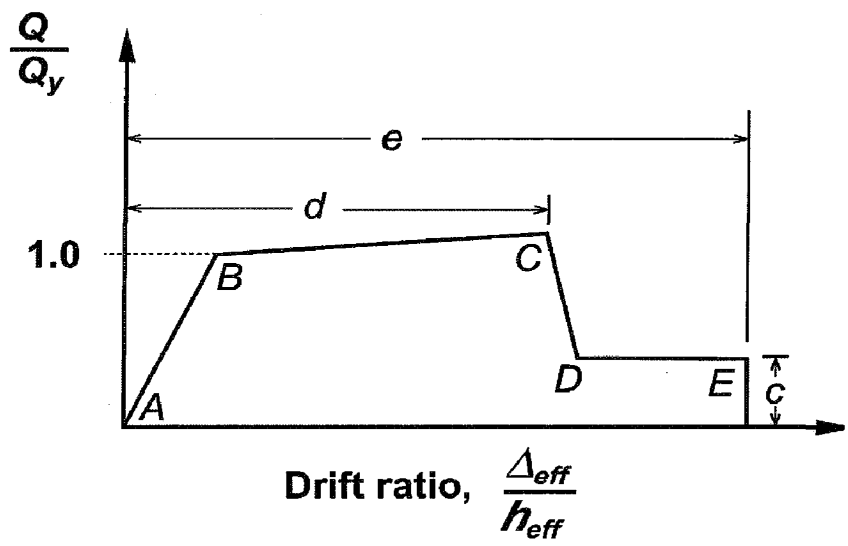

Fig. 6.3 Generalized Force-Deformation Relations for Masonry Elements or Components (FEMA-356)

The required force deformation properties of hinges for each infill panels have been derived from the strength and aspect ratio given in FEMA 356.

The non-linear action of strut with link elements as gap elements, which are active in compression only, is described in Fig. 6.4.

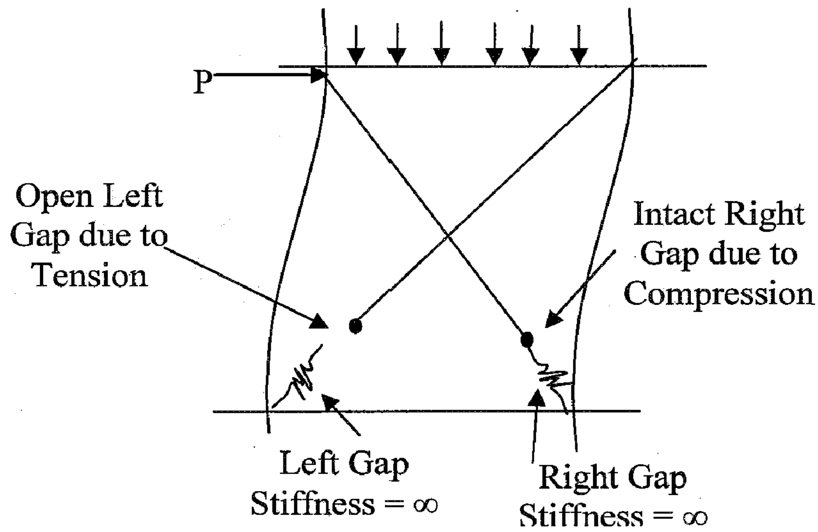

Fig. 6.4 Strut Model of Infill for Non- Linear Analysis under Lateral Load

6.2.2.1 Stage Construction of Infills

In general design practice URM infills are treated as non-structural elements and their stiffness, strength and interaction with frame is often ignored, only weight of infill is taken into design consideration. Again, it is common practice in field that bare frame is first constructed completely then only infills are placed in between the beam-column gap, hence infill dose not contribute in transferring gravity load, as there is no proper contact between beams and infill panel. This behaviour is simulated in Non-Linear Static Pushover Analysis through Non-Linear staged Construction such that no vertical is transferred through infills. This model can capture the frame infill interaction in global sense.

6.3 DYNAMIC CHARACTERISTICS

A comparison between the dynamic characteristics of 4-storey bare frame buildings and infilled frame buildings has been presented in Tables 6.1 – 6.3. It has been observed that time periods get reduced tremendously due to inclusion of infill in structures. A comparison between time periods from analysis results and codal provisions has been made. It has been observed that codal formulae give much less time periods. It imposes larger base shear on the building and consequently larger cross sectional area of frame elements are required. Thus provides conservative design. It has been observed that mass participation is higher in the infilled frame buildings than in the case of bare frame buildings.

Table 6.1 Fundamental Time Periods for 4-storey Bare and Infilled Frame Buildings

Design Level	**Frame Configuration**	**Fundamental Time Period (sec) (From Analysis)**		**Time Period (sec) (From IS: 1893-2002)**	
		Longitudinal	Transverse	Longitudinal	Transverse
Gravity Designed	Bare	1.750	1.971	0.563	0.563
	Infilled	0.403	0.498	0.261	0.353
SMRF, Capped Period	Bare	1.317	1.52	0.563	0.563
	Infilled	0.384	0.458	0.261	0.353

Table 6.2 Modal Mass Participation Factors for Bare and Infilled Frame 4- Storey Buildings Designed for Gravity Load

Mode	Bare Frame			Infilled Frame		
	Time Period (sec)	Modal Mass Participation Factor (%)		Time Period (sec)	Modal Mass Participation Factor (%)	
		Longitudinal	Transverse		Longitudinal	Transverse
1	1.971	0.00	86.46	0.498	0.00	89.12
2	1.789	38.08	0.00	0.403	51.58	0.00
3	1.750	47.27	0.00	0.392	38.27	0.33
4	0.631	0.00	8.41	0.168	0.00	7.63
5	0.587	4.15	0.00	0.137	4.86	0.00
6	0.557	5.04	0.00	0.135	2.01	0.00
7	0.349	0.00	1.99	0.101	0.00	0.88
8	0.334	0.92	0.00	0.086	0.95	0.00
9	0.307	1.18	0.00	0.084	0.04	0.01
10	0.241	0.42	0.00	0.082	0.00	0.16
11	0.235	0.00	0.66	0.078	0.03	0.01
12	0.213	0.30	0.00	0.075	0.02	0.00

Table 6.3 Modal Mass Participation Factors for Bare and Infilled Frame 4- Storey Buildings Designed as SMRF as per Relevant IS Codes

Mode	Bare Frame			Infilled Frame		
	Time Period (sec)	Modal Mass Participation Factor (%)		Time Period (sec)	Modal Mass Participation Factor (%)	
		Longitudinal	Transverse		Longitudinal	Transverse
1	1.520	0.00	83.92	0.459	0.02	88.54
2	1.391	38.78	0.00	0.384	47.90	0.01
3	1.317	44.08	0.00	0.361	40.90	0.13
4	0.477	0.00	9.46	0.157	0.01	7.71
5	0.442	6.30	0.00	0.132	4.62	0.01
6	0.430	4.18	0.00	0.128	2.73	0.00
7	0.252	0.00	2.74	0.095	0.00	1.03
8	0.242	1.12	0.00	0.082	0.85	0.00
9	0.227	1.36	0.00	0.079	0.25	0.00
10	0.164	0.00	1.03	0.076	0.01	0.24
11	0.163	0.98	0.00	0.066	0.25	0.00
12	0.151	0.25	0.00	0.065	0.00	0.00

6.4 YIELD PATTERNS

To understand the failure mechanism of the global structure due to presence of infill and its effect on different design levels, a study of yield pattern has been made under incremental monotonically increased static load at different directions. The yield pattern of 4-storey bare frame buildings and uniformly infilled frame buildings designed for gravity load and the same building designed as SMRF is presented through Fig. 6.5- Fig. 6.12 respectively. It has been observed that for bare frames hinges first occur at shorter span beams in both directions due to concentration of drift demand and again at further pushing, hinges forms in all the ground storey columns after formation of hinges in almost all the beams. On the contrary, in case of uniformly infilled frames hinges first occur at infills as it is attracting much more force due to increased stiffness in comparison of columns.

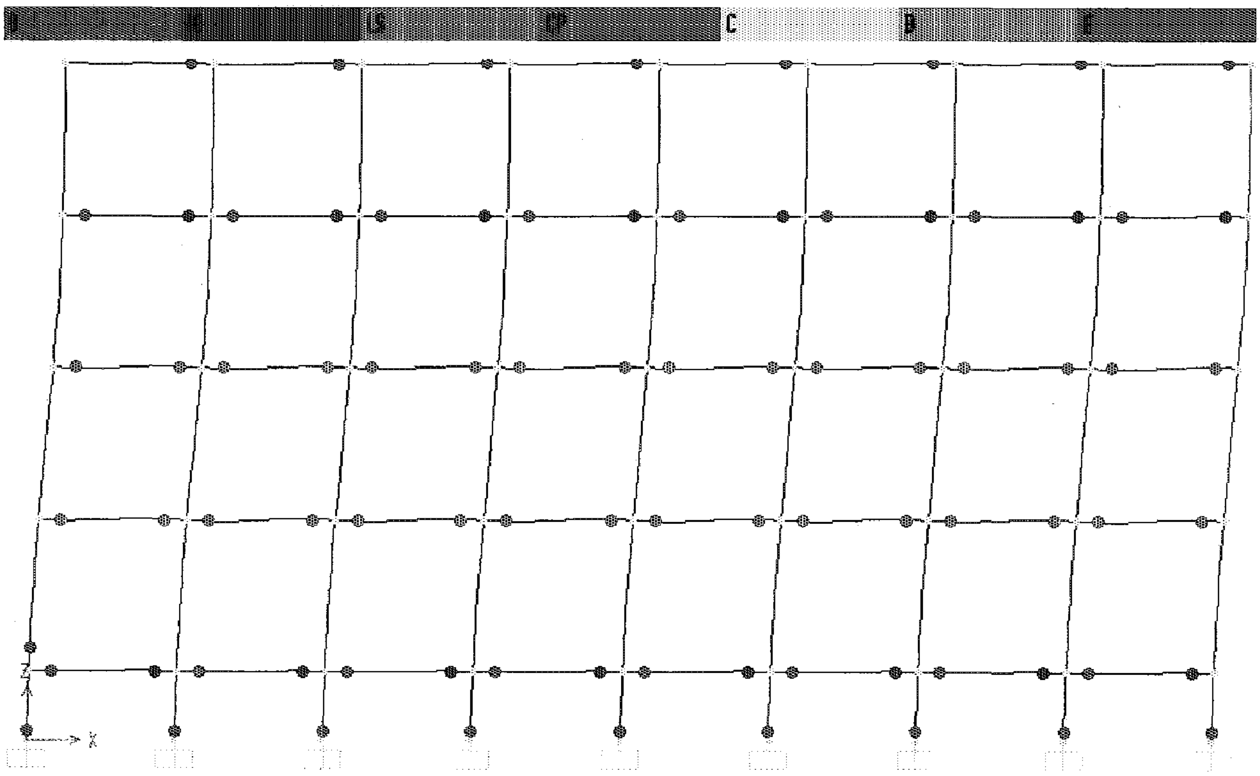

Fig. 6.5 Yield Pattern at Ultimate Point for Bare Frame Building Designed for Gravity in Longitudinal Direction

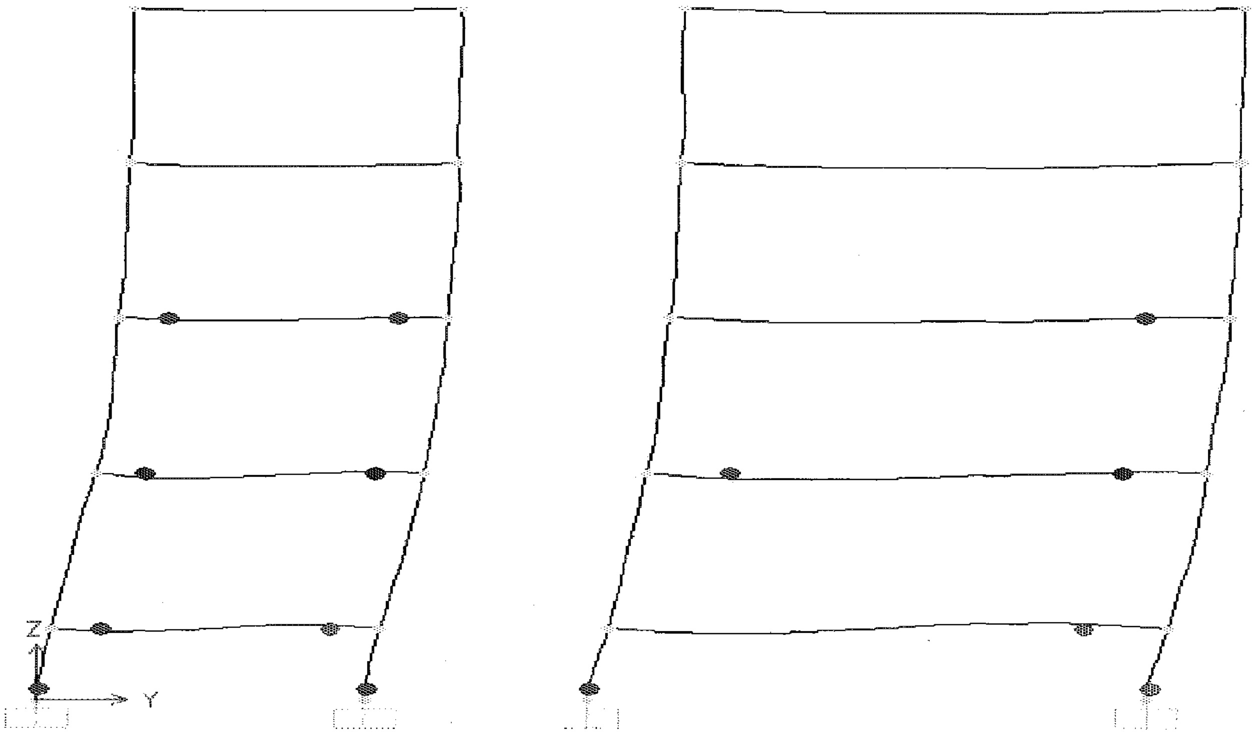

Fig. 6.6 Yield Pattern at Ultimate Point for Bare Frame Building Designed for Gravity in Transverse Direction

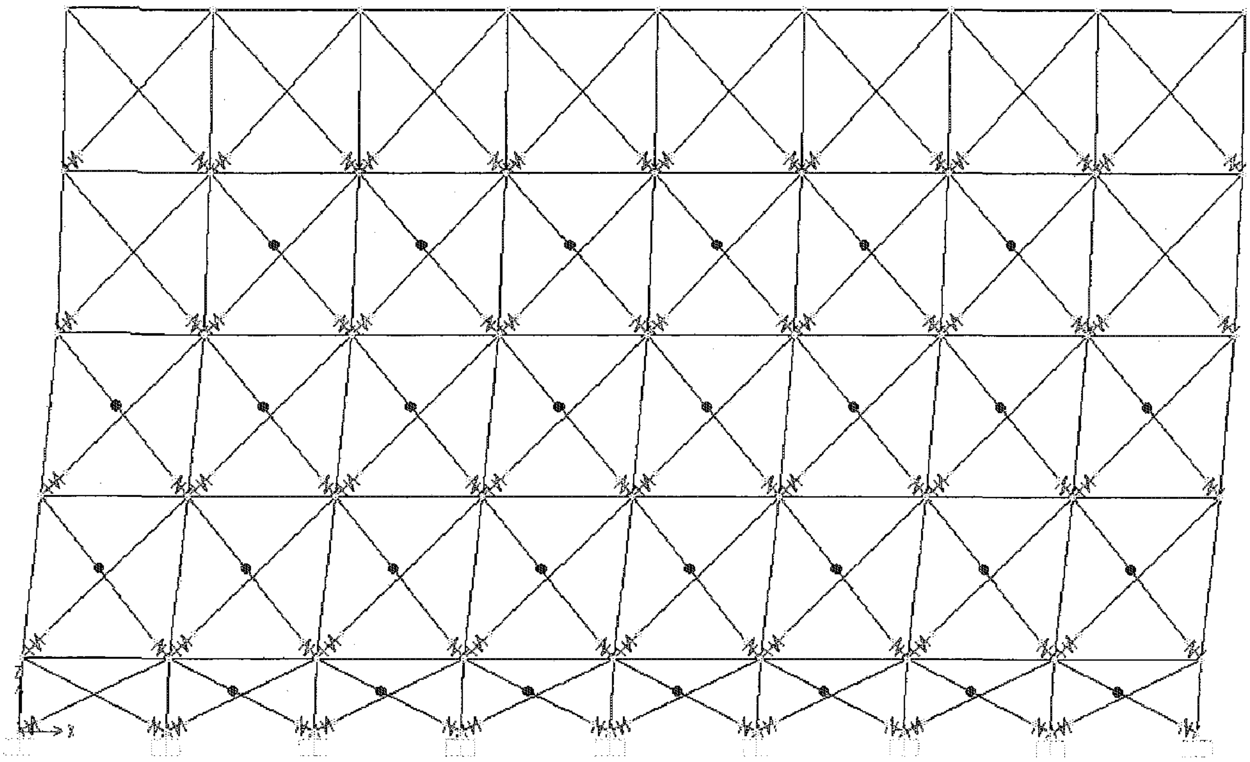

Fig. 6.7 Yield Pattern at Ultimate Point for uniformly infilled Building Designed for Gravity in Longitudinal Direction

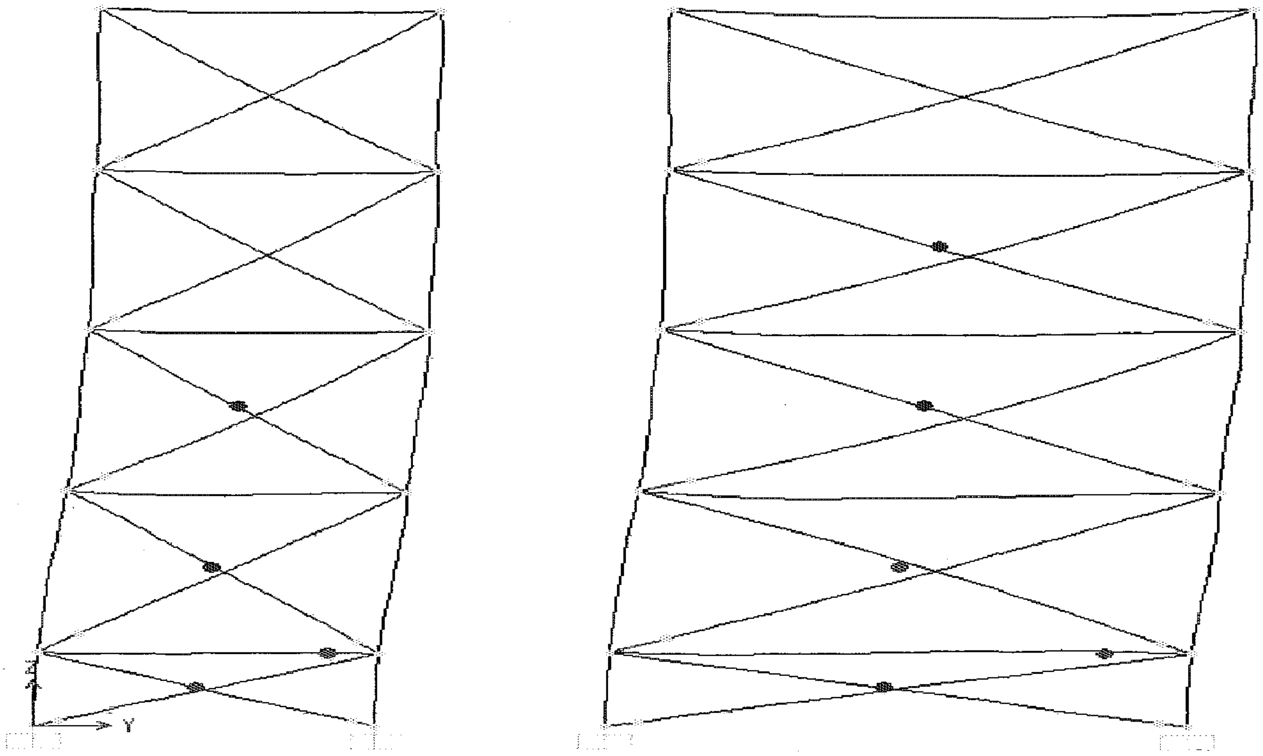

Fig. 6.8 Yield Pattern at Ultimate Point for uniformly infilled Building Designed for Gravity in Transverse Direction

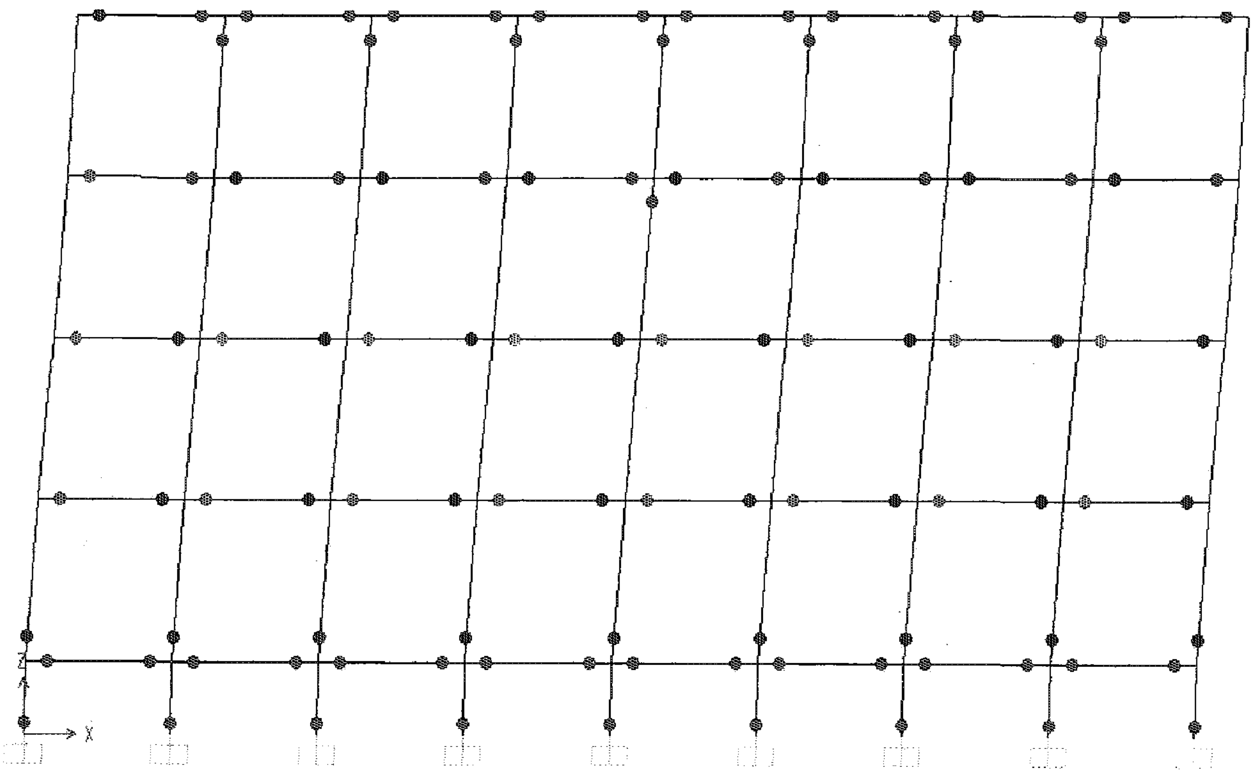

Fig. 6.9 Yield Pattern at Ultimate Point for Bare Frame Building Designed as SMRF in Longitudinal Direction

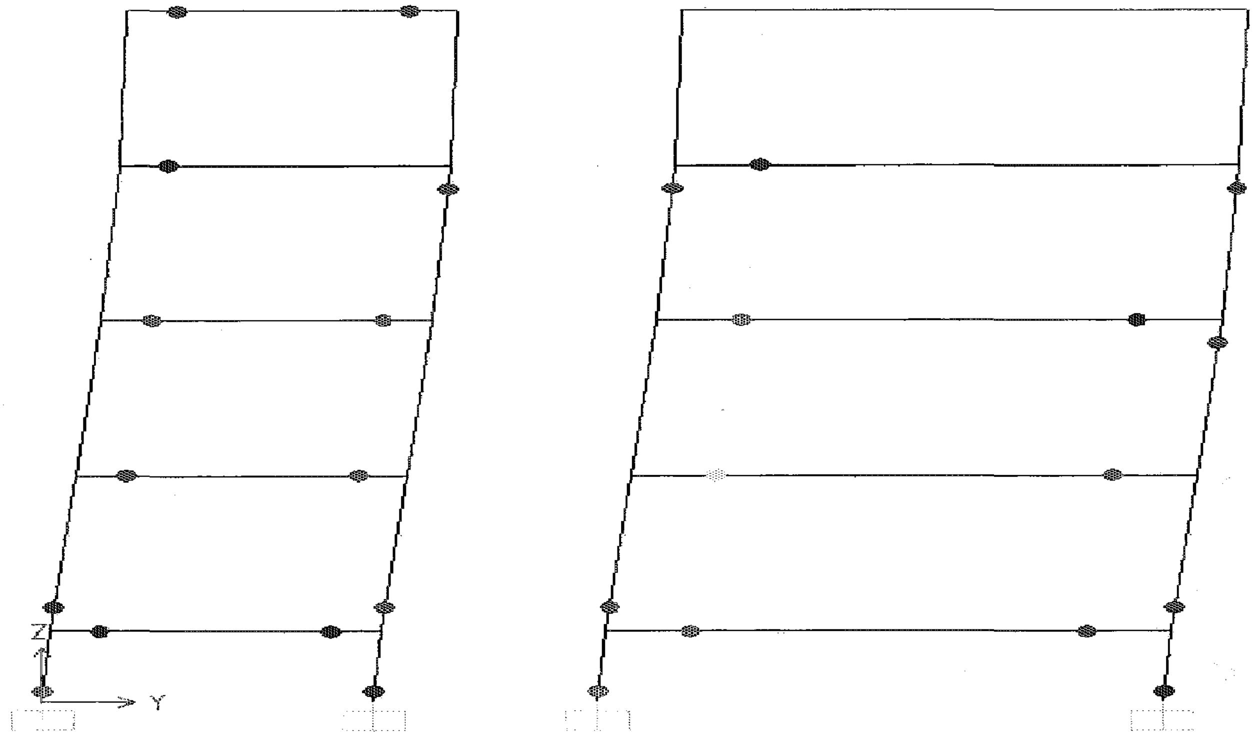

Fig. 6.10 Yield Pattern at Ultimate Point for Bare Frame Building Designed as SMRF in Transverse Direction

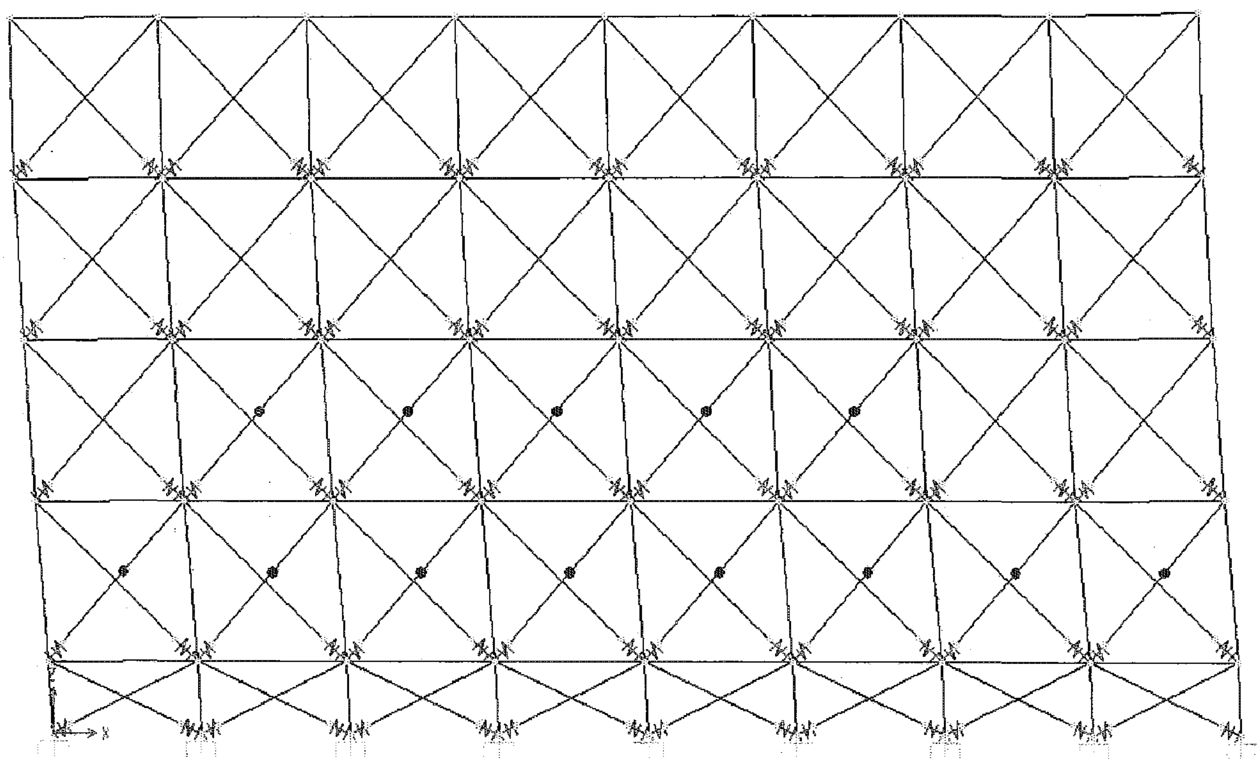

Fig. 6.11 Yield Pattern at Ultimate Point for 4-Storey uniformly infilled Building Designed as SMRF in Longitudinal Direction

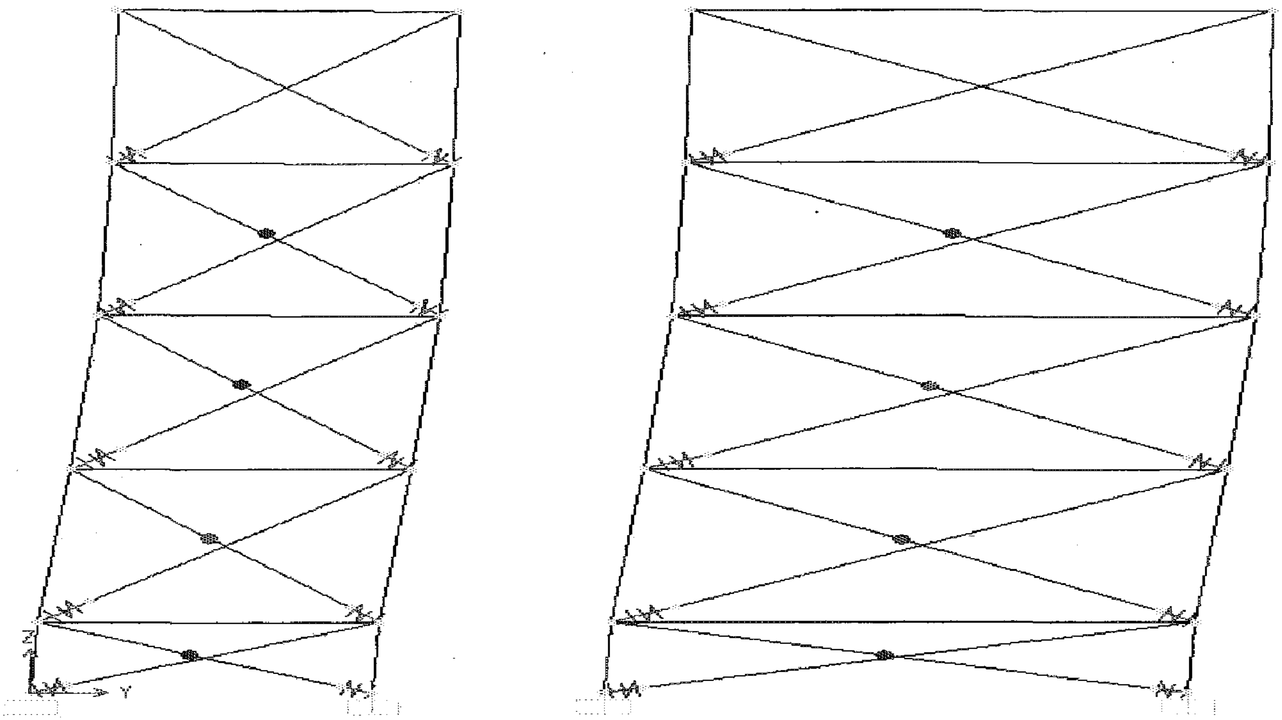

Fig. 6.12 Yield Pattern at Ultimate Point for 4-Storey uniformly infilled Building Designed as SMRF in Transverse Direction

6.5 PERFORMANCE LEVELS

A comparison of performance levels of bare frame and uniformly infilled frame buildings designed for gravity load and with seismic design as per relevant IS codes have been shown in Fig. 6.13 – Fig. 6.16. It is clearly visible from the capacity curves that the stiffness of infilled frame buildings is much higher than that of the bare frame buildings. However, performance level of infilled frame buildings has been found at higher base shear and lower roof displacement compared to its bare frame counterparts. As the building is exactly symmetrical in longitudinal direction the capacity curves are perfect mirror image for the cases where loading is subjected from positive and negative both directions [Fig. 6.13, Fig. 6.15], but the curves slightly differ in case of loadings in transverse direction [Fig. 6.14, Fig. 6.16]. In each capacity curve, black dot (•) represents the Performance Point for DBE and black triangle (▲) represents the Performance Point for MCE; the three crosses (x) represent IO, LS and CP Performance Levels, consecutively.

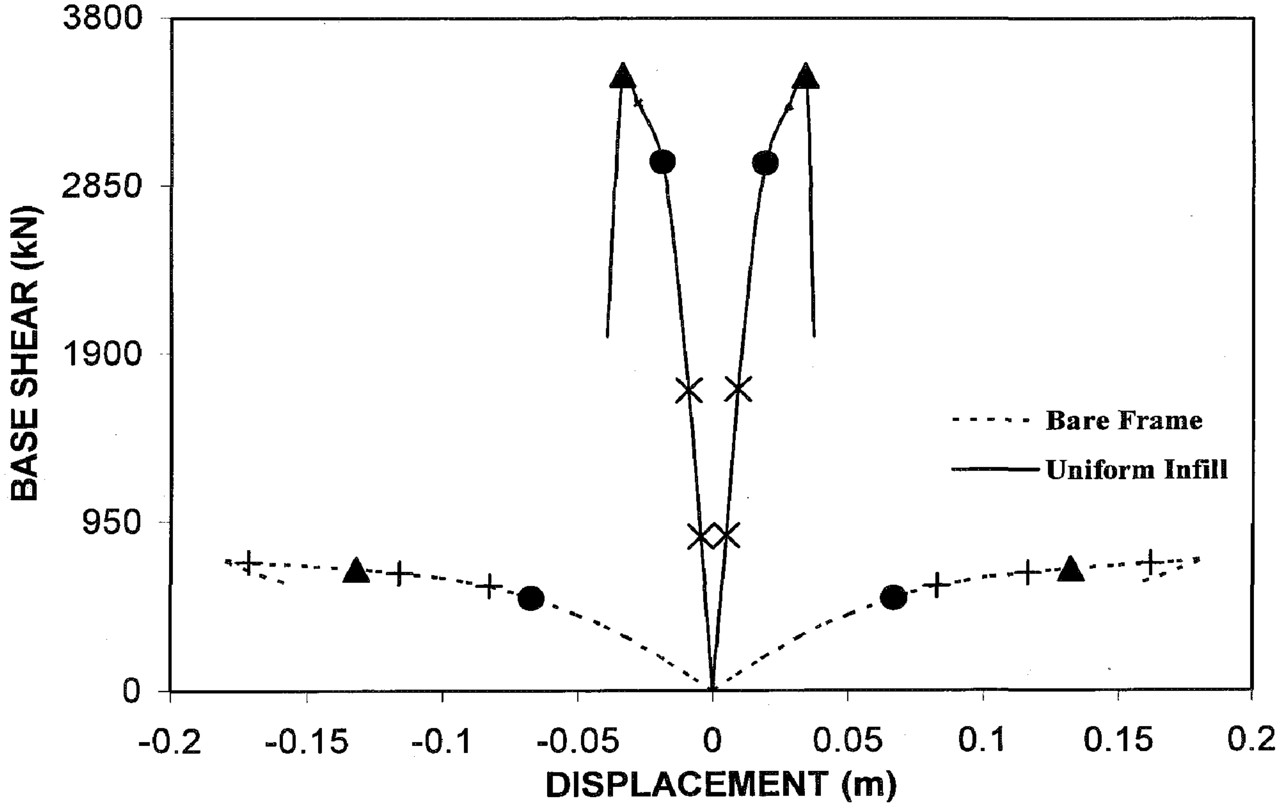

Fig. 6.13 Capacity Curves for 4 – Story Bare Frame and Uniformly Infilled Frame Building Designed for Gravity Load; in Longitudinal Direction

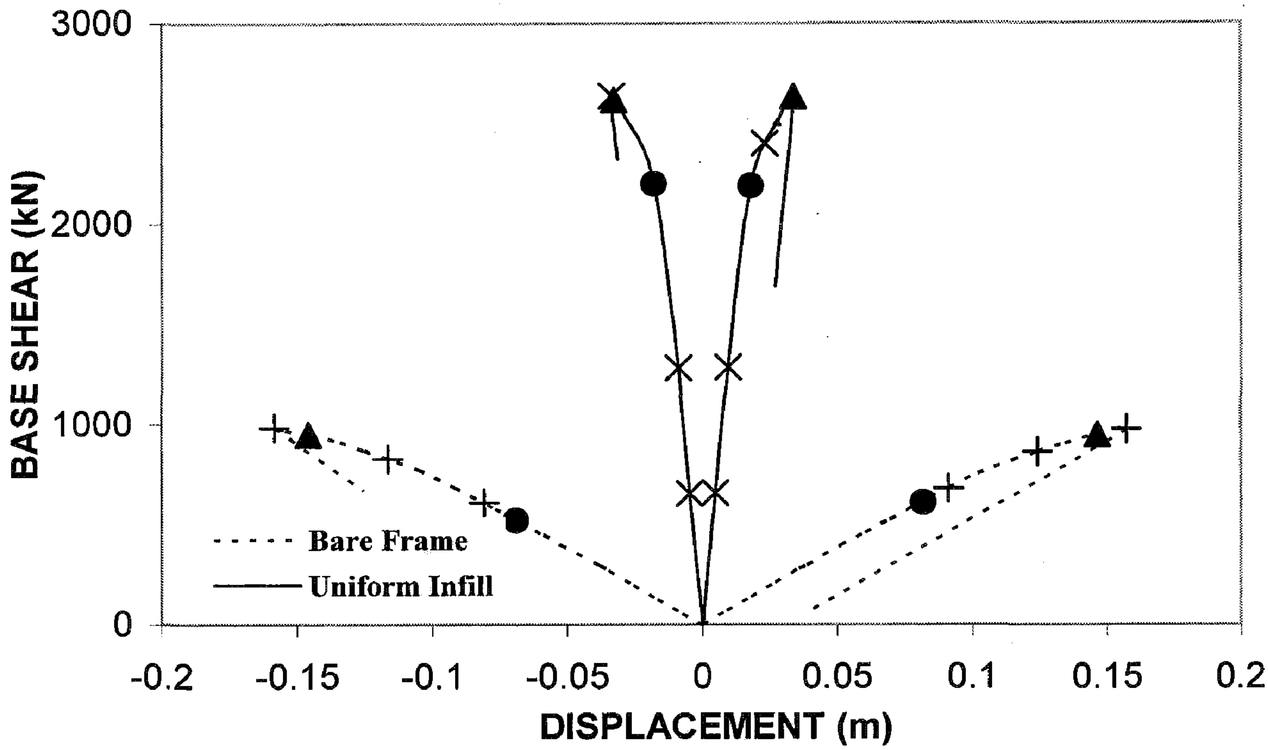

Fig. 6.14 Capacity Curves for 4 – Story Bare Frame and Uniformly Infilled Frame Building Designed for Gravity Load; in Transverse Direction

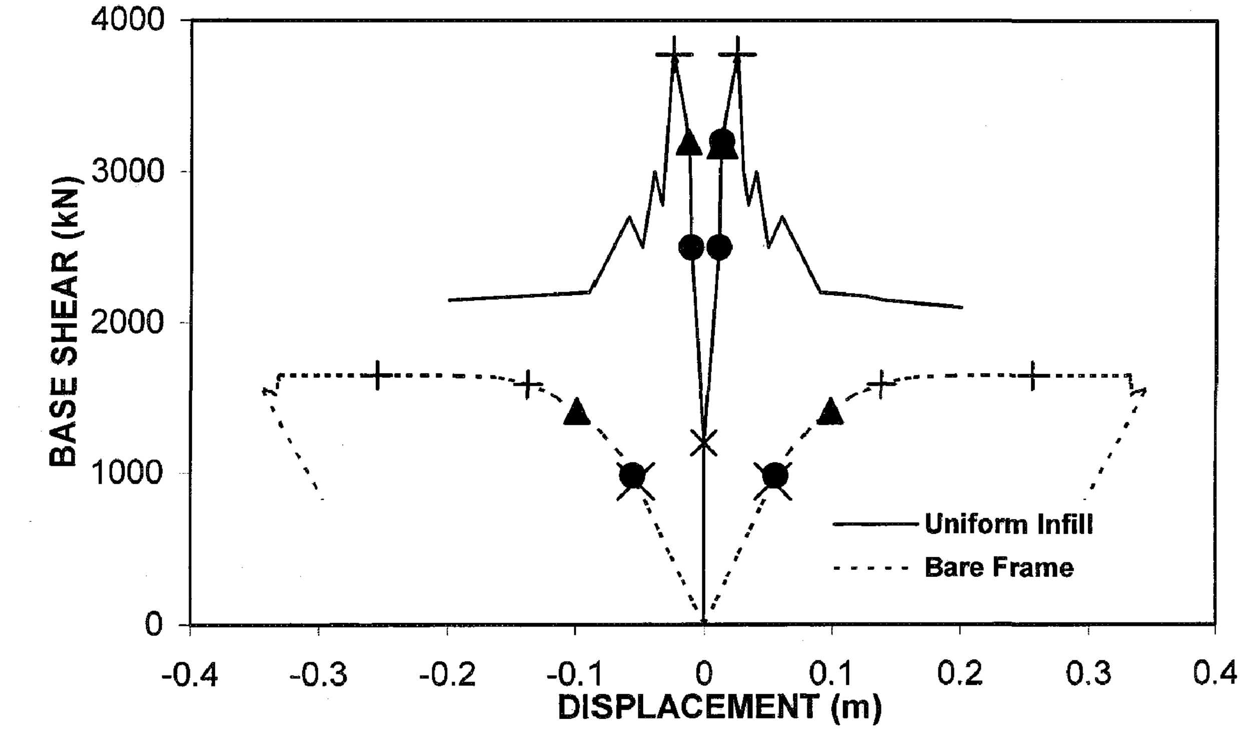

Fig. 6.15 Capacity Curves for 4 – Story Bare Frame and Uniformly Infilled Frame Building Designed as SMRF, as per Relevant IS Codes; in Longitudinal Direction

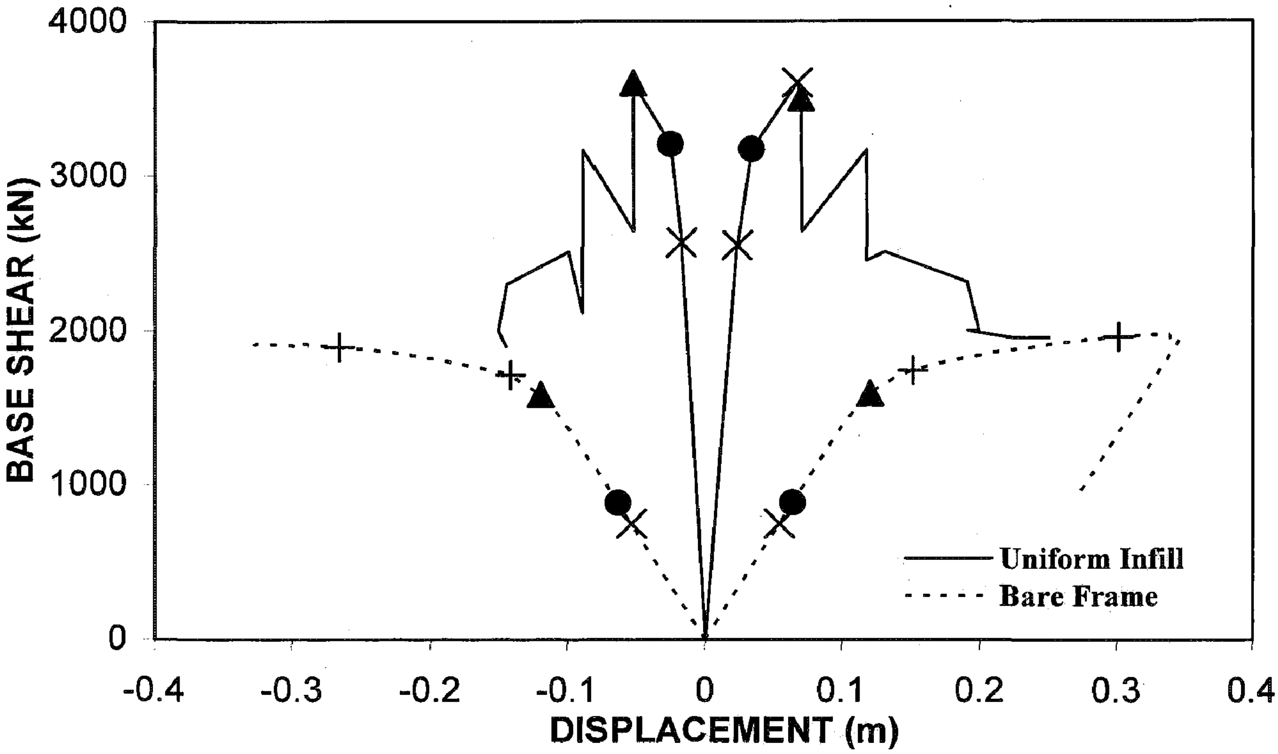

Fig. 6.16 Capacity Curves for 4 – Story Bare Frame and Uniformly Infilled Frame Building Designed as SMRF, as per Relevant IS Codes; in Transverse Direction

6.6 VULNERABILITY ASSESSMENT

Vulnerability of bare frame building and the same building designed with uniform infill has been accessed to study the Effect of URM infill on the vulnerability of RC frame buildings, presented in Fig. 6.17 and Fig. 6.18 through vulnerability/fragility curves. Vulnerability curves are derived using HAZUS methodology described in section 5.3. Uncertainties in capacity spectra, demand spectra, damage thresholds are handled as per Table 5.2 in CHAPTER 5. Table 6.4 show the median spectral displacement values used to construct the fragility curve and DPMs. Fragility curve showing the cumulative probability of being in a particular damage state is calculated using equation (5.2). Table 6.5 shows the damage probabilty of the of 4-storey bare frame buildings than that of the same building designed with uniform infill considering the strength and stiffness of infill, for design PGA (0.12g) and higher.

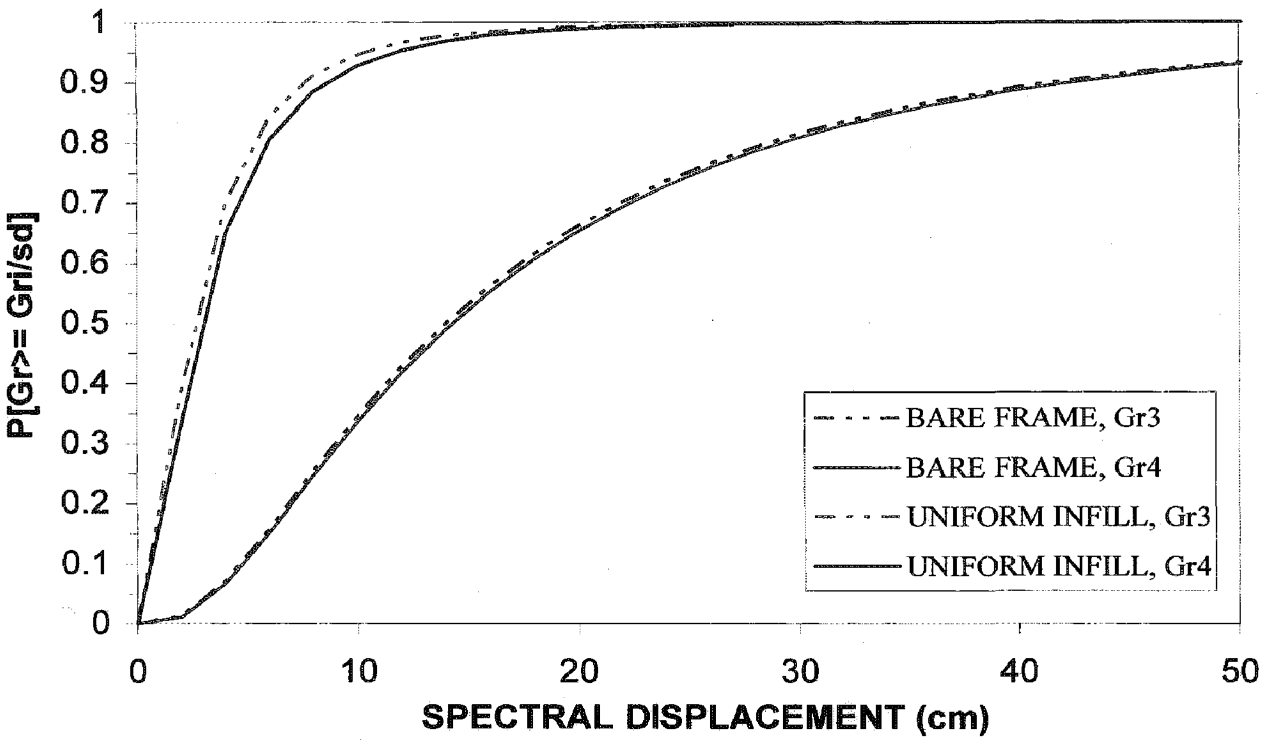

Fig. 6.17 Comparison of Vulnerability for Damage Grades Gr3 and Gr4 of 4-storey Bare Frame and Uniformly Infill Frame Building Designed for Gravity Load Only

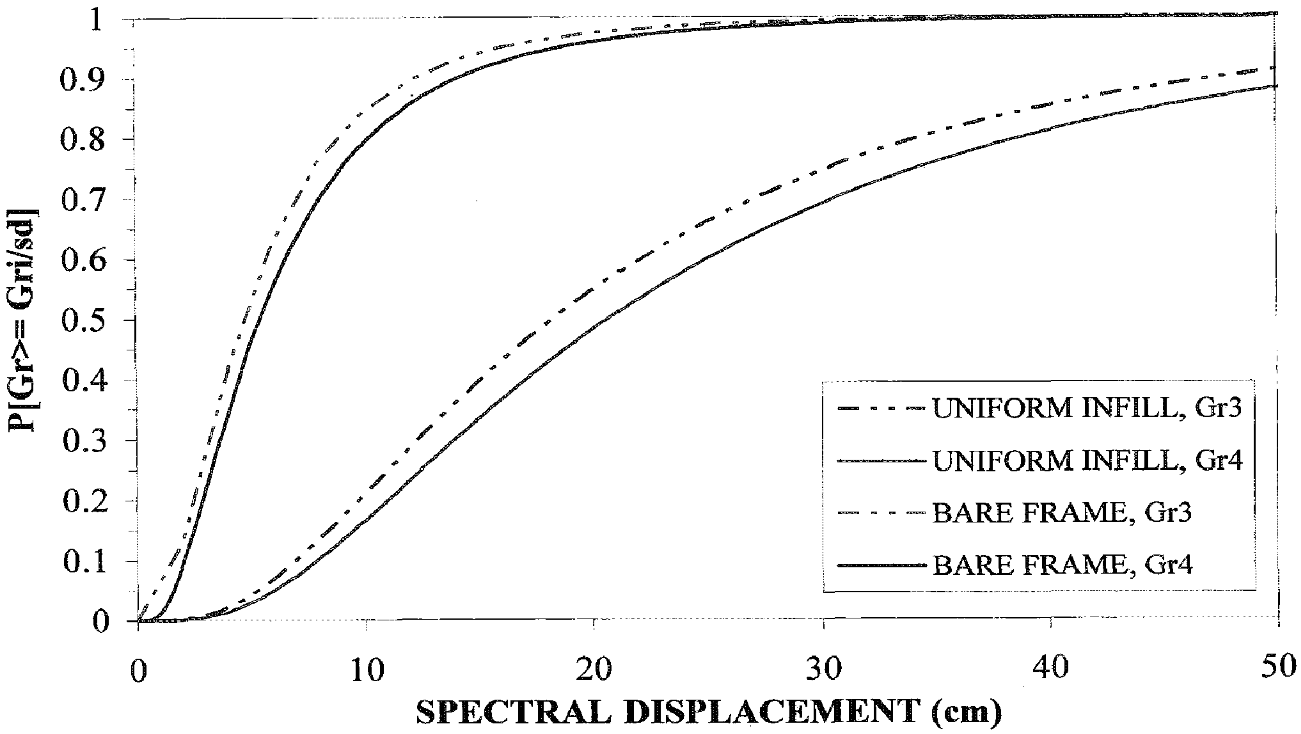

Fig. 6.18 Comparison of Vulnerability for Damage Grades Gr3 and Gr4 of 4-storey Bare Frame and Uniformly Infilled Frame Building Designed as SMRF, as per Relevant IS Codes

Table 6.4 Median Spectral Displacement for 4-Storey Bare Frame Buildings and Uniformly Infilled Buildings

Design Level	Damage Grade, Gr1.	Damage Grade, Gr3.	Damage Grade, Gr4.
	Median S_d (cm)	Median S_d (cm)	Median S_d (cm)
Gravity Designed Bare Frame	5.32	10.77	10.97
Gravity Designed Infill Frame	0.89	1.96	2.22
SMRF Bare Frame	7.09	14.93	16.00
SMRF Infill Frame	1.61	3.62	4.19

Table 6.5 Damage Probabilities (%) for 4-Storey Bare Frame Buildings and Uniformly Infilled Buildings

Design Level	Damage ≥Gr.1 for PGA (g)				Damage ≥Gr.3 for PGA (g)				Damage ≥Gr.4 for PGA (g)			
	0.12(g)	0.18(g)	0.24(g)	0.36(g)	0.12(g)	0.18(g)	0.24(g)	0.36(g)	0.12(g)	0.18(g)	0.24(g)	0.36(g)
Gravity Designed Bare Frame	51.96	70.06	80.64	91.01	21.73	38.05	51.37	69.54	12.79	37.25	50.53	68.80
Gravity Designed Infilled Frame	68.62	83.20	90.33	96.23	32.79	51.25	64.42	80.14	20.04	45.38	58.80	75.79
SMRF Bare Frame	24.55	44.11	59.31	78.12	4.63	12.69	22.43	41.41	5.87	10.86	19.76	37.84
SMRF Infilled Frame	36.17	57.41	71.58	86.67	7.58	18.58	30.50	51.22	2.08	13.81	24.03	43.45

6.7 DISCUSSION

The dynamic characteristics of bare frame changes significantly due to inclusion of infills. Time periods get reduced substantially due to presence of infills. It is observed that in case of uniformly infilled frame buildings, hinges occur first in infills and then in beams and columns and finally the building becomes unstable due to formation of hinges in ground storey columns. It can be concluded form the Capacity Curves and Performance Points of the two systems that in case of uniformly infilled frame buildings, Ductility Capacity reduces tremendously while the Strength Capacity increases, as compared to that of the bare frame as the URM infills have high strength and stiffness but very low ductility. Again, the fragility curves of bare frame and uniformly infilled frames indicate higher damage probability of infilled frame than that of bare frame for both gravity designed buildings and building designed as SMRF, as per relevant IS codes. Gravity designed infilled frame buildings have the worst performance and maximum damage probability for all the grades of damages. SMRF infilled frames are having more threat than SMRF bare frames but the damage probability is much less than that of gravity designed infilled frames.

CONCLUSIONS AND RECOMMENDATIONS FOR FUTURE WORK

7.1 CONCLUSIONS

Parametric study has been performed on a set of multistoreyed RC frame buildings to assess the efficacy of the different design provisions of the IS codes and to study the effect of different design considerations on the anticipated performance and seismic vulnerability of the buildings. Effect of URM infills on the Dynamic Characteristics, Seismic Performance, and Seismic Vulnerability of RC frame buildings has also been studied and the following conclusions are made:

1. It can be observed from Pushover curves that earthquake forces govern the design in the present case (Seismic Zone – IV) for 4 storey, as well as, for 9 storey buildings.
2. Consideration of earthquake effects, as per IS:1893, in the design increases the strength and ductility capacities of the building significantly and this effect increases with increase of building height.
3. Drift limits as specified in IS:1893, govern the design of SMRF and OMRF frames in most of the cases.
4. In most of the cases, considered in this study, the buildings designed and constructed properly for the gravity loads alone, have survived the earthquakes up to those specified for Seismic Zone IV. This shows the significant overstregth available in case of code designed buildings.
5. Buildings designed in accordance with the present seismic code requirements for DBE, show IO level Seismic Performance for MCE, in most of the cases.
6. SMRF buildings have enhanced ductility but reduced strength, as compared to OMRF buildings. The performance of SMRF buildings has been observed to be only marginally better than OMRF buildings.

7. Seismic Vulnerability greatly depends on the design level and probability of damage increases with increase in building height for all grades of damages.

8. Gravity designed building pose worst damage threat although it has possessed considerable overstrength and ductility to survive against collapse at MCE.

9. Buildings designed as SMRF as per IS:1893 and IS:13920 pose higher probability of damage in lower damage state (Gr.1) than buildings designed as OMRF as per IS:1893 for a particular PGA.

10. Damage probability calculated with different uncertainty for different design levels, sometimes gives inconsistent results, although it is not rational to assume same uncertainty factor for all design levels.

11. Damage probability of higher damage grades (Gr.3 and Gr.4) of SMRF design is quite close to that of OMRF. This is because the allowable drift limits in IS:1893 have been defined at design force level, which means that the SMRF, is designed for total drift 5/3 times higher than that of OMRF. It has been shown that if limit is applied on total drift, as in case of IBC and EC8, the damage probability for SMRF is much less than that for OMRF.

12. URM infills have drastic effect on the Dynamic Characteristics, Global Failure Mode, Strength and Ductility of RC frame Buildings. The Strength and Stiffness is remarkably increased but the Ductility is drastically reduced. The overall effect is significant deterioration in Seismic Performance of the buildings.

13. The fragility analysis shows that the Infilled frames are much more vulnerable than bare frame designed for the same level of earthquake forces, as per the relevant IS codes.

14. The deterministic framework of Performance Based Design does not provide the complete insight into the Seismic Performance and risk associated with the designed buildings. A probabilistic framework is necessary to get the complete picture.

7.2 RECOMMENDATIONS FOR FUTURE WORK

Present study is based on analytical simulation of the seismic behaviour, which needs to be validated by experimental results. Therefore large scale tests of bare and infilled RC frames are required to be undertaken. Further, variabilities in different input parameters have been adopted from HAZUS. These variabilities in Indian constructions need to be evaluated using extensive field studies. Further, there are several deficiencies in buildings which crop up during construction. The effect of these deficiencies can also be simulated based on field studies.

In the present study, uniformly placed URM infills have been considered with in-plane modelling. The effect of asymmetry, openings, and out of plane failure of infills can be further studied.

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