IN A REGULAR RC FRAMED BUILDING MOMENT CAPACITY RATIO AT BEAM COLUMN JOINT

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ABSTRACT

Reinforced concrete moment resisting frames (RCMRF) are structural systems that should be designed to ensure proper energy dissipation capacity when subjected to seismic loading. In this design philosophy the capacity design approach that is currently used in practice demands "strong-column / weak-beam" design to have good ductility and a preferable collapse mechanism in the structure. When only the flexural strength of longitudinal beams controls the overall response of a structure, RC beam-column connections display ductile behaviour (with the joint panel region essentially remaining elastic). The failure mode where in the beams form hinges is usually considered to be the most favourable mode for ensuring good global energy-dissipation without much degradation of capacity at the connections. Though many international codes recommend the moment capacity ratio at beam column joint to be more than one, still there are lots of discrepancies among these codes and Indian standard is silent on this aspect. So in the present work pushover analysis is being done using SAP 2000 for increasing moment capacity ratio at beam column joints and its effect on the global ductility and lateral strength of the structure is studied. To incorporate the uncertainties in material properties, a probabilistic approach is followed to observe the effect of ground motion intensity on probability of exceedance of any specific damage state for structures designed considering different moment capacity ratios (MCR) at the connections. For this objective fragility curves are developed considering the pushover curves obtained from the nonlinear static analysis. Ductility of the structure increases with increase of MCR. Also the buildings designed with lesser MCR values are found to be more fragile compared to the building with higher MCR.

Keywords: Pushover, Moment Capacity Ratio, Fragility, Ductility, Lateral Strength

I. INTRODUCTION

Earthquake is a global phenomenon. Due to frequent occurrence of earthquakes it is no more considered as an act of God rather a scientific happening that needs to be investigated. During earthquake, ground motions occur both horizontally and vertically in random fashions which cause structures to vibrate and induce inertia forces in them. Analysis of damages incurred in moment resisting RC framed structures subjected to past earthquake show that failure may be due to utilization of concrete not having sufficient resistance, soft storey, beam column joint failure for weak reinforcements or improper anchorage, column failure causing storey mechanism. Beam-column connection is considered to be one of the potentially weaker components when a structure is subjected to seismic loading. Figures of some of the beam column joint failure and column collapses in past earthquakes Along with the development of many strength-based design procedures, currently used performance-based seismic design approach of building includes the capacity design philosophy proposed by Paulay and Priestley (1992) as an important tool for earthquake resistant design. In this process the design is based on both the stress resultants obtained from linear structural analysis subjected to code specified design lateral forces and equilibrium compatible stress resultants obtained from pre-determined collapse mechanism. The flexural capacities of members are determined on the basis of overall structural response of a structure to earthquake forces. For this purpose, within a structural system the objects which can be permitted to yield before failure otherwise known as ductile components and the objects which will remain elastic and will collapse immediately without warning known as brittle components are chosen.

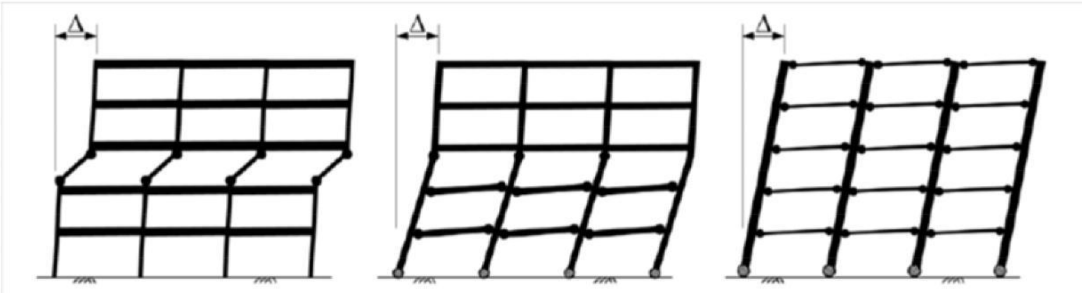
II. STRONG COLUMN WEAK BEAM DESIGN (SCWB)

Designing a building to behave elastically during earthquake without any damages will make the project uneconomical. So the earthquake-resistant design philosophy allows damages in some predetermined structural components. One of the most important requirements of the building to withstand any type of earthquakes is not the more force it can resist but the more deformation it can take before complete collapse. Capacity design procedure sets strength hierarchy first at the member level and then at the structure level. So, it needs adjusting of column strength to be more than the beams framing into it at a joint. Mathematically it can be expressed as ≥ Where M_c and M_b are moment capacities at the end of column and beam meeting at a joint respectively. The reasons for adopting this SCWB design are discussed below:

Failure of column will lead to global failure of the structure but if there is flexure failure in beam ends still it can carry gravity loads because its shear capacity is not hampered. The beam has to support the floor but column has to take the weight of entire building above it. So failure of the column is more critical than the beam failure.

Beam with lesser compression loads on them can be designed to be more ductile than columns and absorb large amount of energy through inelastic actions. As the maximum level of displacement loading that may come during an earthquake loading is not known beforehand so the building should be designed such that the ductile that is the under reinforced flexure failure mode precedes the brittle (shear) or non-ductile mode of failure. As in ductile chain analogy it is observed that if all the links are brittle and one is ductile and a relative displacement is applied at the ends, internal forces are developed in the links and ultimately the chain breaks when the link with least strength breaks. If this weakest link is ductile type then the chain undergoes large final elongations before fail. So to make a chain ductile the weakest link is made ductile. Similarly to make a structure ductile the weakest component should be the beam as the under reinforced flexure failure and less axial force in beam make it more ductile than columns.

concentrate in one or a few stories only (Fig. 1.a), and if the drift capacity of the columns is exceeded then it is of greater consequence. but, if columns provide a stiff and strong spine over the building height, drift will be more uniformly distributed thus reducing the occurrences of localized damages.(Fig)



(a) Storey Mechanism (b) Intermediate Mechanism (c) Beam Mechanism

III. BACKGROUND AND MOTIVATION

In ACI web sessions 1976, when the structure detailed in Fig. 1.4 was being tested for checking the type of joint failure an unexpected result obtained and the beam failed instead of the failure at joint. While investigating this issue the column to beam moment capacity ratio (refer Eq. 1) obtained was more than one.

Moment capacity ratio (MCR) = $\underline{\sum^{\Sigma}}$

Where M_{nc} = flexural strength of columns framing into joint and M_{nb} = moment capacities of beams framing it. Hence this concept of moment capacity ratio came into picture. Column–beam flexural strength ratio is certainly an important variable for consideration in overall frame performance. It also determines whether it is t Many international design codes recommend that design flexural capacity of columns framing into the joint is greater than design flexural capacity of beam framing into it. According to some of these codes this ratio varies from 1 to 2. But failure of numerous code-compliant buildings during past earthquake by formation of storey mechanism raises concern the capacity of beam or column that will establish the input force for which joint is designed.

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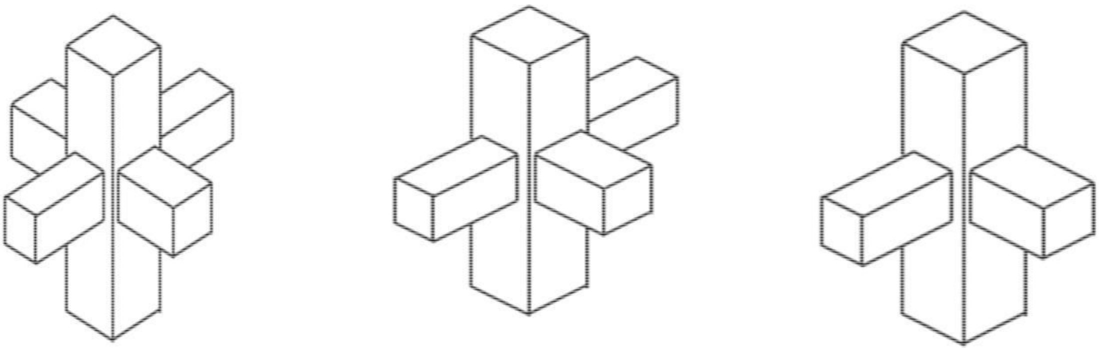
IV. PUSHOVER ANALYSIS

Performance based design philosophy includes the determination of two quantities for design and analysis purpose, one is seismic demand and the other is seismic capacity. Seismic demand is the effect of earthquake forces actually coming on the structure and seismic capacity is the ability of the structure to resist earthquake forces. The performance is evaluated in such a manner that capacity should be more than the demand.There are so many methods for inelastic structural analysis like linear static analysis, linear dynamic analysis, nonlinear static and nonlinear dynamic analysis procedure. Linear methods are used when the nonlinearity expected is low indicated by demand capacity ratio (DCR) value less than 2.0. Due to some limitations and disadvantages of other methods nonlinear static analysis or push over analysis is considered as the most suitable method for performance based seismic design because it requires less effort and deals with less amount of data for the analysis purpose. Static analysis is proved to be accurate when the structure is short and regular in plan so that higher modes effect are less significant otherwise dynamic analysis is also to be used along with static method. The use of non-linear static analysis in seismic engineering dates back to the research of Gulkan and Sozen (1974). He used a single degree of freedom system (SDOF) to represent equivalently a multi-degree of freedom structure. The load-displacement curve obtained from this substitution to the real structure was found out by either finite element analysis or manual calculation to obtain the initial as well as post-yield stiffness, the yield strength and the ultimate strength. For analysing multi-degree of freedom systems simplified non-linear inelastic analysis procedures have also been proposed by Saudi and Sozen (1981) and Fajfar and Fischinger (1988).In the context of describing recent advanced developments in earthquake resistant design, Krawinkler (1995) discussed non-linear static pushover analysis that is, in most cases the normalised displacement profile obtained at a first estimate of the target displacement level is used for defining the shape vectors. The most simple and practical work in this context is done by Sasaki *et. al.*(1996). This work includes running of multiple pushover analyses under forcing vectors corresponding to number of modes that are excited during dynamic response. Out of various modes the mode which will be more effective and cause maximum damage were known if the individual pushover curves converted to capacity curves between spectral displacement versus spectral acceleration using the dynamic characteristics of the individual modes. This procedure is instinctive, and gives advantage over conventional single mode pushover analysis in identifying potential problems. A brief review done by Tso and Moghadam (1998) showed that fixed load patterns used in pushover analysis have certain disadvantage, but there is not sufficient research to prove the newly proposed variable load patterns as a better option. Kim and D’Amore (1999) compared pushover analysis with inelastic time history analysis. They concluded that all analyses of a structure under a set of specific earthquake motions are not predicted by pushover analysis, a rather obvious conclusion that did not require inelastic dynamic analysis to prove. A single push over analysis under a predefined or fixed loading pattern or displacement vector may not be sufficient to describe the interaction between the continuously-varying dynamic characteristics of an inelastic multi-degree of freedom structure with the various set of natural frequencies.A modified procedure for pushover analysis was discussed by Bracci *et.al.* (1997). It consists of analysing the structure assuming a fixed lateral pushover load pattern usually triangular. Subsequent increments in loads are calculated from the instantaneous storey resistance and the base shear obtained in the previous step. This method is used in defining the moment curvature relationship of the various members which is used as an input parameter and is utilised to capture the effect of local response. Effect of contribution of higher modes is neglected. Rana *et. al.*, (2004) performed pushover analysis of a 19-storey building in San Francisco where the plastic rotations of the hinges developed, as calculated by SAP2000, were checked and found to be within the limits suggested by FEMA and ATC guidelines for the intended design objective of Life Safety. The performance of RC frames was investigated using pushover analysis by Sugani *et. al.*,2012. They observed the performance point *i.e.*, the intersection of seismic demand and capacity curves. Distribution of damages in beams and columns are observed and for properly detailed RC building most of the hinges were formed in beams.Kihara *et. al.*(2008) observed the column to beam strength ratio of 30 multi storied buildings and concluded that when steel frames are subjected to strong-axis ground shaking, maximum storey drift angle increases for the column-beam strength ratio more than √2.Poluraju *et. al.*(2011) performed pushover analysis of RC framed structure and concluded that the performance of properly detailed reinforced concrete frame building is adequate as indicated by the performance point and the distribution of Hinges in members show that the desirable collapse mechanism was obtained. Most of the hinges developed in the beams and very few in columns with limited damage. The causes of failure of reinforced concrete building frames during seismic loading may be due to poor quality of the materials .The results obtained in terms of demand, capacity and plastic hinges gave an insight into the real behaviour of structures

V. FRAGILITY ANALYSIS

Seismic damage assessment can be done by means of fragility curves. Various approaches for damage evaluation, like first-order reliability and a fuzzy random method, were reviewed by F.Colangelo (2008). These methods were used for deterministic analysis of infill reinforced-concrete frame, and the fragility curves obtained were compared. It was concluded that randomness of the capacity is essential. If any damage state is assessed with a deterministic drift range, then fragility sharply increases with increase in peak ground acceleration of ground motion and hence it is quite overestimated.Seismic fragility curves of RC buildings are studied by Alexander Papailia (2011). The analysis performed for the estimation of the peak response quantities was according to Eurocode 8 (Parts 1 and 3) with certain simplifying assumptions for the frames. The design and the evaluation of the building performance was based on the results of linear elastic (equivalent) static analysis, for a lateral force pattern that is distributed over the height as per an assumed linear mode shape, termed as “lateral force method” in Euro code 8. The median value for the probabilistic distribution corresponding to a given damage state and damage measure of interest was obtained. The dispersion (β value) of the fragility curve was taken into account explicitly the model uncertainty for the estimation of the damage measure given the intensity measure and the uncertainty of the capacity in terms of the damage measure.Sharfuddin *et. al.* (2010) evaluated the COF requirement that ensure probabilistically the preferable entire beam hinging failure mode and avoid undesirable storey collapse modes of the frame structure during earthquakes. It is found that under same reliability level target COF requirement increases with number of storey and it decreases with the increase in the reliability level.Milutinovic &Trendafiloski (2003) developed the fragility curves for buildings considered within the RISK-UE project. Building classifications were taken from the stock in the project application sites, *i.e.*, Barcelona, Bitola, Bucharest, Catania, Nice, Sofia and Thessaloniki. Both empirical as well as analytical procedures were employed. Different damage scales were considered for each approach and a correlation between them was proposed. For steel, wood and reinforced masonry structures, the HAZUS vulnerability curves were adopted. Empirical curves obtained using damage probability matrices which were generated from vulnerability index that accounts for structural characteristics and local conditions.Kappos *et. al.* (2003), within the RISK-UE project, used capacity spectrum method on various configurations of regular RC framed structures designed considering contribution of in-fill and without in-fills (the case of soft ground storey was also examined) for different seismic design levels. Due to assumption of bilinear response in the capacity spectrum method, when there is higher displacement demand than the capacity of regularly in-filled frames with good seismic design, use of the capacity curve for bare frame is recommended. The uncertainty associated in definition of different damage states and the different variability of the capacities were taken from HAZUS. The dispersion for all damage states of a given structural class was the mean of the dispersions for each damage state, so as to obtain non-intersecting fragility curves. Reference was made to the cost of replacement and to a damage index. The vulnerability curves were developed following the hybrid method, where analytical and observational capacity curves are combined. Akkar *et. al.* (2005) developed vulnerability curves for low-rise and mid-rise infill framed RC buildings. Pushover analyses were performed on 32 existing buildings in Duzce to define the intervals of base shear capacity , period and ultimate drift for two, three, four and five storey buildings with low-level of seismic design. Nonlinear dynamic analyses were performed for 82 recorded accelerograms and bilinear structures with properties within the selected intervals. The effect of number of stories was found significant on the probability of exceeding the moderate and the severe damage limit states. Instead of peak ground acceleration Spectral displacement can be better correlated with peak ground velocity, specifically for higher levels of damage. There was good agreement of the vulnerability curves with observed damage after the 1999 Duzce earthquake. Erberik (2008) considered 28 RC framed buildings constructed between 1973 and 1999 that were inspected after the Düzce earthquake. Bilinear capacity curves were obtained from non-linear static (pushover) analysis by the distribution of their characteristic properties. 2800 nonlinear dynamic analyses of randomly sampled SDOF structures were also performed for a set of 100 recorded accelerograms. The effect of post-yield to initial stiffness ratio variability (negligible), simulation (negligible) and sampling (negligible) techniques, sample size (negligible), limit state variability (important) and degrading hysteretic behaviour (important) was studied by means of parametric analyses. The analytical curves predicted were in good agreement with the observed damages.Rossetto and Elnashai (2005) developed fragility curves for low-rise infill RC frames, that were designed and detailed according to Old Italian seismic code. Building design and modelling inputs, uncertainties in the material properties used like randomness in concrete, steel and masonry properties and ground motions data chosen were representative of that region. Adaptive pushover analysis was conducted and a trilinear idealisation of the capacity curve obtained for infill frames were considered. Rather than using graphical method, Nonlinear dynamic analysis was performed for the equivalent SDOF structure and the performance point was obtained. Ten accelerograms records were selected to account for ground motion variability (Wen & Wu, 2001). The results of these analyses were used to construct response surfaces, from which the vulnerability curves were developed considering random values of material properties and spectral displacement to obtain the maximum drift. The analytical curves were in reasonable agreement with empirical curves.



(a) Interior joint **(b) Exterior joint** **(c) Corner joint**

Fig. : Typical Diagram Showing Beam-Column Joint

In an exterior joint one beam and two columns are framing into the joint. So, one beam moment is distributed to two columns. Hence the relation between moment capacities of members framing into an exterior joint is as given in equation (2.7):

$$(\ + \ ) \geq \eta \, M(B + 0) \ldots\ldots \quad (2.7)$$

In a corner joint one beam and one column are framing into the joint. So the one beam moment is distributed to one column. Hence the relation between moment capacities of members framing into a corner joint is as shown in equation (2.8):

$$(\ + 0) \geq \eta \, M(B + 0) \ldots\ldots \quad (2.8)$$

Therefore keeping one constant factor as MCR for all the joints may result in predominant ground storey column failure, which is not preferred.

There are a lot of discrepancies among all the codes regarding the flexural capacity of column to beam at the joint. IS code does not include any such criteria for strong column weak beam design. So it is very important to propose a MCR suitable for Indian Standard

For steel building lots of research are being done in this area in past. But for RC buildings there are not sufficient literature in this area.

VI. METHODOLOGY

a) Five, seven and ten storey RC framed (Plane) buildings are designed using commercial software STAAD-Pro.
b) Ultimate flexural capacity of beam ($M_{r,b}$) is determined from the design data obtained.
c) Column reinforcement in the buildings is progressively increased to attain different column to beam moment capacity ratio (MCR) at maximum moment, at zero axial load and at design axial load.

VII. BUILDING DESIGN AND MODELLING

The present study is based on analysis of a family of reinforced concrete multi-storeyed building frames. These buildings were first designed using STAAD-Pro. The input data required for the design of these buildings are presented in Table1 (a-c).

Table 1(a) General Building and Location Details

Type of structure	Multi storey RC frame
Zone	V
Exposure Conditions	mild
Soil type	medium
Damping	5 %
Storey height	3m
Bay width	4m
Design philosophy	Limit State method conforming to IS 456:2000

Table 1b Details of Materials and Section Property

Beam	300mm□ 300mm
Column	300mm□400mm
Concrete	f_{ck}= 25 MPa Poisons ratio = 0.3 Density = 25 kN/mm^3 Modulus of elasticity = 5000 $\sqrt{f_{ck}}$ =25000 MPa
Steel	f_y = 415 MPa Modulus of elasticity = 2□10^{5}MPa

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Table 1(c) Loadingdetails for the Design

Dead load	20 kN/m
Live load	10 kN/m
Equivalent lateral loads	as per IS 1893 (Part I):2002

7.1 Steps used in Pushover Analysis

1. The building is modelled using SAP2000 and the hinge properties are defined and assigned as per FEMA 356 and ATC 40 guidelines.
2. First gravity pushover is applied incrementally under force control for the combination of DL+0.25LL.
3. Then lateral pushover is applied that starts after the end conditions of gravity push over under displacement control to achieve the target ultimate displacement or final collapse.
4. The lateral load pattern to be used in the pushover may be in the form of a specified mode shape, uniform acceleration in specified direction, or a user defined static load case. Here the distribution of lateral force employed is in form of the first mode shape *i.e* the structure is going to vibrate in its fundamental mode.
5. In the model, beams and columns were modelled using frame elements, into which the hinges were inserted. Diaphragm action was assigned to the floor slabs to ensure integral lateral action of beams in each floor.
6. The structure is pushed until global collapse is reached that means when sufficient numbers of plastic hinges are formed to develop a collapse mechanism under the target displacement.

7.2 Building Capacity Curve

Capacity curve can be represented as a plot of lateral resistance of a structure as a function of lateral displacement. It is derived from the pushover curve obtained from nonlinear static analysis in the form of base shear versus roof displacement . Capacity curve for every model building can be obtained for different level of design for a given loading condition and specific performance level. The yield and ultimate spectral displacements are obtained from the bi-linearization of capacity curves. Yield capacity indicates the lateral strength of a structure. The yield spectral displacement can be taken as the displacement where significant loss of stiffness of the structure occurred due to yielding of maximum number of members. Up to the yield point the curve is linear with stiffness depending on expected time period of a structure. From yield capacity to ultimate capacity there is change of slope indicating the development of plastic stage. After the ultimate point the building's lateral load resisting capacity decreases appreciably. The ultimate spectral displacement is taken as the maximum displacement where strength of a structure decrease to 85 % of the peak strength as considered in this present study. Figure shows a typical building capacity curve as per HAZUS. Where C_s is point of significant yielding of design strength coefficient; T_e is the expected elastic fundamental mode period of the building; $\square_1$ is fraction of building weight effective in pushover mode; γ is the over strength factor relating true yield strength to design strength ;λ is over strength factor relating ultimate strength to yield strength and μ is the ductility ratio relating ultimate displacement to λ times yield displacement

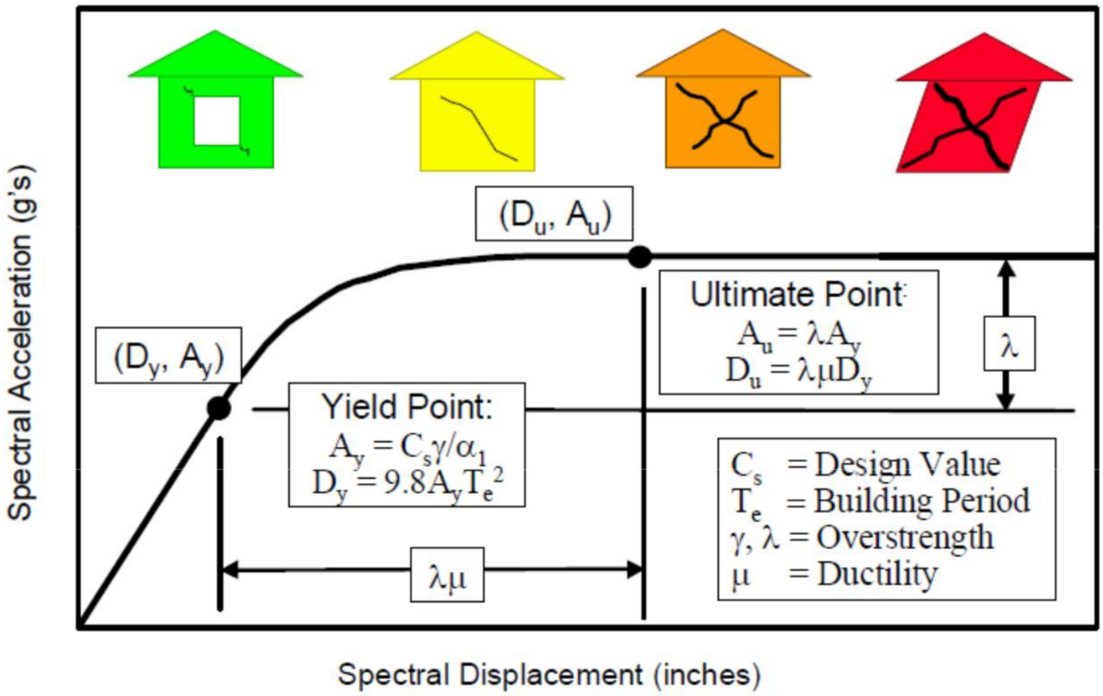

Fig. Typical Building Capacity Curve with Performance (HAZUS, 2003)

Plane building frames are designed with IS-456:2000 for loading requirement of IS-1893:2002 and IS-875 (Part-1, 2) using STAAD-Pro for varying MCR.

Nonlinear static analysis is being carried out to understand the effect of MCR in the response of framed building.It is found that with increase of MCR at design axial load upto 1.47 for uniaxial bending in a plane frame improves the ductility at an expense of extra reinforcement, with further increase of MCR there is not much increase in ductility. Increase in strength either at yield or maximum is not very significant with progressive increase in MCR for a seven storey building frame. but for 5 storey and 10 storey frames strength also increase significantly upto MCR 1.7.Since seismic design philosophy aims to achieve good ductility in a structure so we need not have to think for higher strength but for higher ductility.A preferable collapse mechanism can be achieved by increasing MCR.

7.3 Scope of Future Study

The analysis can be extended with considering more number of buildings with different varying parameters.

Here only regular RC framed buildings are considered. The analysis can be extended for irregular building having torsion effects.

Only internal joints are considered in the present work. For external and corner joints also analysis can be done.

Effect of infill wall can also be evaluated in the analytical models.

The ground motion parameter can be selected not only as spectral displacement but also in terms of PGA or PGV etc.

By taking more MCR values the analysis can be done for more number of buildings.

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