

Moment Capacity Ratio at Column – Beam Joint in a RC Framed Structure

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Abstract-Beam-column joint is the gap in the modern ductile design of building. Especially under the earthquake loading this is more susceptible to damage. Due to brittle nature of failure this type of failure cannot be afford. Since 1970's this areas is under the light of research, but with the paper of Park and Paul, It got momentum. But still due to versatile nature of the joints core behaviour, the problem is still persisting. Though many international codes recommend the moment capacity ratio at beam column joint to be more than one, There are discrepancies among the major international codes with regard to MCR. Indian standard codes for design of RC framed buildings are silent on this aspect. Draft 13920 (2014) code suggests a value of MCR similar to other international codes without proper theoretical basis. Hence a rational study is required on the values of MCR. A computationally attractive procedure for calculating flexural capacity of column developed for determining MCR at a beam-column joint. To reach at an appropriate and acceptable MCR for capacity design of RC framed building reliability based approach is done. This research deals with the fragility and reliability analysis of five, seven and ten storey RC frames designed using various values of MCR ranging from 1.0 to 3.2. The RC frames are designed as per IS 1893 (2002) for all seismic zones. Hazard curves required of various seismic location in India (like zone II, III, IV and V) has been selected from National Disaster Management Authority, Government of India. Seismic risk assessment of all the designed buildings is conducted and based on the achieved Reliability Index and the Target Reliability Index minimum value of Moment Capacity Ratio (MCR) is suggested.

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 [1]. 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 [2-4]. Figures of some of the beam column joint failure and column collapses in past earthquakes are shown in Fig. 1. Therefore such column and joint failure need to be given special attention.

Fig. 1. Failure of column with eccentric connection



during Turkey earthquake

II. LITERATURE REVIEW

Sugano *et. al.*,(1988) conducted experimental programme on 30-storey RC framed building in Japan and developed

design consideration to ensure good collapse mechanism and also observed the ductility of plastic hinges.

Nakashima (2000) observed for steel building that the column over strength factor increases with increase in ground motion amplitude for ensuring column-elastic response. Also for frames in which column-elastic behaviour is ensured, the maximum story drift angle is 1.5 to 2.5 times as large as the maximum overall drift angle.

Hatzigeorgiou (2009) performed an extensive parametric study on inelastic behaviour of reinforced concrete frames under reverse cyclic ground motions and observed the relationship as shown in equation (1)

$$\sum M_{n,c} \geq 1.3 \sum M_{n,b} \quad (1)$$

Jain et. al., (2006) proposed that, when a reinforced concrete moment resisting frame is subjected to seismic loads, at beam-column joint, summation of moment of resistances of columns should be greater than or equal to 1.1 times summation of moment of resistance of beams framing into it as in equation (2)

$$\sum M_{n,c} \geq 1.1 \sum M_{n,b} \quad (2)$$

In this seismic design concept, it is assumed that beams yielding in flexure will precede possible yielding of columns which is recognized as the favorable failure mode.

III. OBJECTIVE

Based on the discussions presented above the main objective for the present research is defined as follows:

- To observe the effect of MCR on ductility and strength of a structure.
- The effect of MCR on probability of failure of multi-storeyed building.
- To arrive at a moment capacity ratio suitable for Indian Standard.

IV. 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.

- d) Considering the beam and column reinforcement, the same building is modelled using SAP2000 and nonlinear static analysis is being done.

A. Pushover Analysis

From the design of doubly reinforced beam using STAAD, ultimate moment capacity of beam obtained for the five storey building, $M_b = 136$ kNm.

By keeping the beam reinforcement fixed the column reinforcements are increased progressively and buildings are modelled using SAP2000 [5].

Although the hinge properties can be calculated using the reinforcement details by using the concrete models such as confined Mander's model, in the present work the force deformation criteria for hinges developed by ATC 40 and FEMA 273 for concrete and steel have been used in pushover analysis.

Basically a hinge represents localised force-displacement relation of a member through its elastic and inelastic phases under seismic loads [6-8]. For example, a flexural hinge represents the moment-rotation relation of a beam of which a typical one is as represented in figure 3.1. M-

θ_p relation for a section consists of plastic rotation and corresponding moments as ratio of yield moment. This relation affects the behaviour of a section when a hinge is formed there. In the present study all values needed to define M- θ_p relations are obtained by following FEMA guidelines.

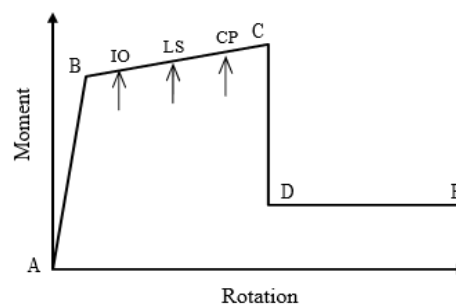


Fig. 2 Typical force-deformation curve showing performance levels

As shown in Fig. 2, five points labelled as A, B, C, D, and E are used to define the moment rotation behaviour of the hinge and three points labelled as IO, LS and CP indicate

the acceptance criteria for the hinge. (IO, LS and CP denote Immediate Occupancy, Life Safety and Collapse Prevention respectively).

B. Point A denotes unloaded condition.

Point B corresponds to yielding of the element. The portion A to B shows linear response of the structure. The slope of B to C portion is very small and it represents strain hardening phenomenon.

The ordinate at C represents ultimate strength and abscissa at C corresponds to the deformation at which significant strength degradation begins.

The drop from C to D represents the initial failure of the element and resistance to lateral loads beyond point C is usually unreliable. D corresponds to the residual strength.

The residual resistance from D to E allows the frame elements to sustain gravity loads. Beyond point E, gravity load can no longer be sustained and the strength of the components is reduced to zero. It corresponds to maximum deformation capacity with the residual strength [9].

The performance of any structure during earthquake depends on the performance of combination of structural and non-structural components [10]. The FEMA 273 defines three structural performance levels and acceptance criteria that relates the earthquake-induced forces and deformations in the structure directly depend on these performance levels which are basically three types as

- 1) Immediate Occupancy (IO)
- 2) Life Safety (LS)
- 3) Collapse Prevention (CP)

C. 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.

D. Pushover Analysis Output

The main output of pushover analysis is in the form of base shear versus roof displacement curve called pushover curves. This capacity curve is generally constructed to represent first mode response of the structure assuming that fundamental mode of vibration is predominant. This assumption holds good for structures with fundamental period up to about one second. For more flexible building with fundamental period greater than one then effect of higher modes should be considered. The pushover curves for 5-storey, 7-storey and 10-storey framed buildings are shown in Figs. 3,4,5

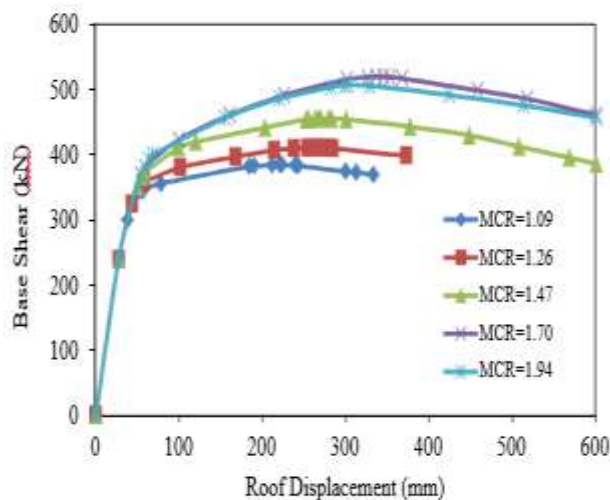


Fig. 3. Pushover curve for 5 storey building frame

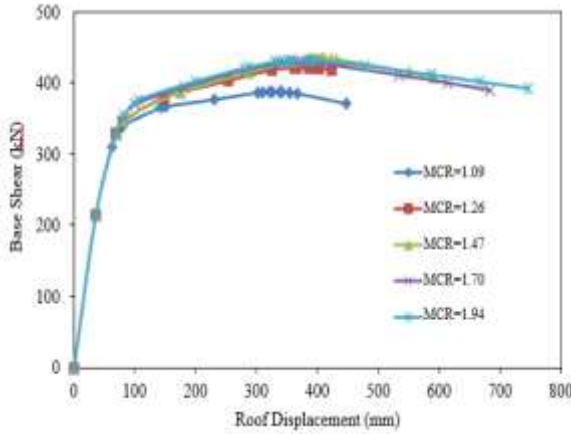


Fig. 4. Pushover curves for 7-storey building

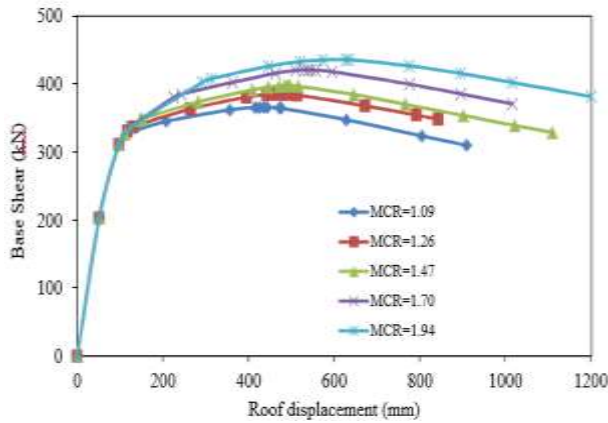


Fig. 5. Pushover curves for 10 storey building

V. RESULTS

A. Performance of 5-Storey 3-Bay Building Frames

Fragility curves for 5-storey 3-bay framed building is developed as per methods discussed above for different MCR values for the two damage states Gr3 and Gr4.

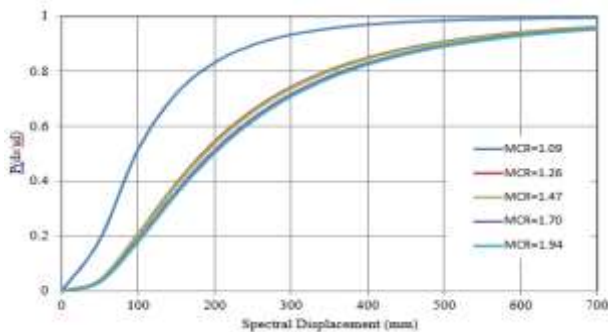


Fig. 6. Fragility curves for 5-storey framed building for Gr3 damage state

The slope of fragility curve developed depends on the log normal standard deviation value. Smaller value of indicates lesser variability of damage state and hence steeper fragility curve is generated. So the Gr3 curves are stiffer than Gr4 curves (Gr3 = 0.75 and for Gr 4 it is 0.85).

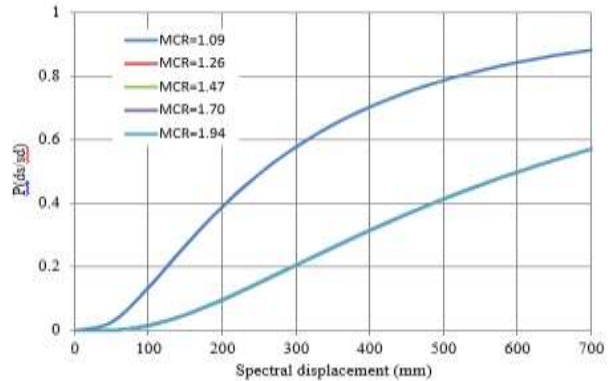


Fig. 7. Fragility curves for 5-storey framed building for Gr4 damage state

B. Performance of 7-Storey 3-Bay Building Frames

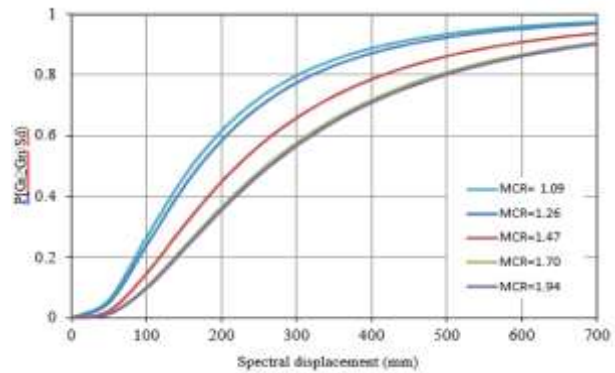


Fig. 8. Fragility curves for 7storey building for Gr3 damage state.

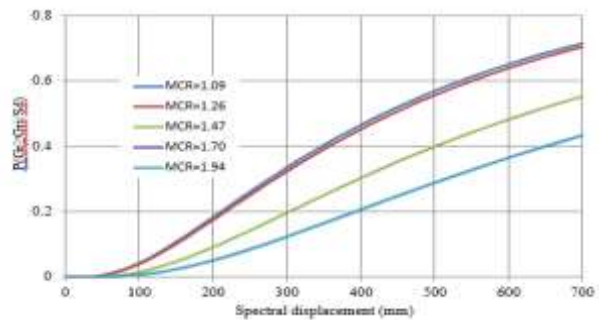


Fig. 9. Fragility curves for 7storey building for Gr4 damage state

C. Performance of 10-Storey 3-Bay Building Frames

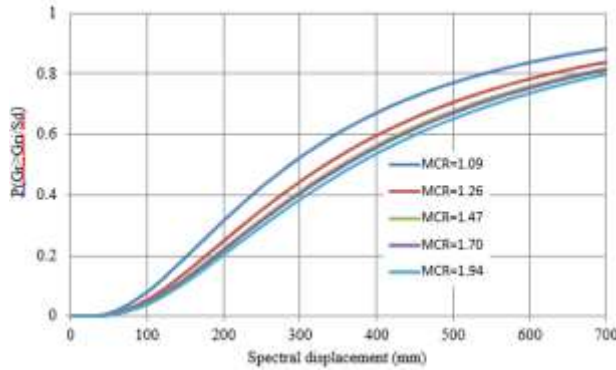


Fig. 10. Fragility curves for 10-storey building for Gr3 damage state

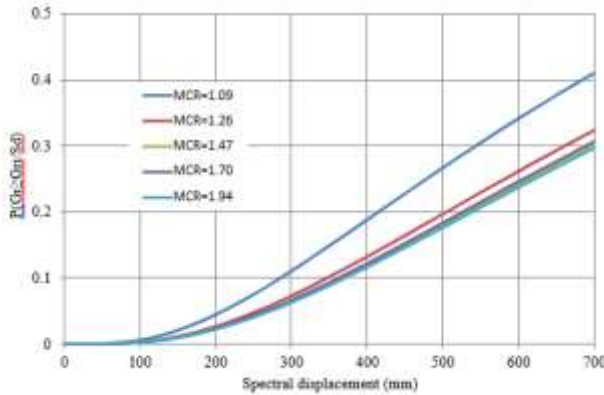


Fig. 11. Fragility curves for 10-storey building for Gr4 damage state

VI. CONCLUSIONS

The conclusions obtained from this research work is summarized into two parts. The first part deals with the pushover analysis and the second part includes fragility analysis.

A. Conclusions from Pushover analysis

The non-linear static analysis is done for 5-storey, 7-storey and 10-storey building frame with increasing MCR.

- 1) For the 5-storey building ductility of the structure increases up to MCR 1.47. Increase of MCR beyond 1.47 does not contribute in enhancing ductility.
- 2) For the 7-storey building ductility increases continuously with increase of MCR up to 1.94. The rate of increase of ductility is higher indicated by the steeper portion of the curve up to MCR 1.47. After MCR 1.47 the rate of increase in ductility decreases.

- 3) The 10-storey building also shows ductility increases up to MCR 1.47 and then it decreases for MCR 1.7. The decrease is may be due to active utilisation of concrete in resisting moment rather than steel. Then for MCR 1.94 ductility again increases. However, highest ductility is achieved at MCR 1.47.
- 4) Lateral Strength of the buildings is showing higher values for MCR 1.7 in most cases for both yield strength and maximum strength condition. But since seismic design philosophy demands deformation based design so ductility is most important parameter than strength.
- 5) So from the pushover analysis it can be concluded that MCR 1.47 seems to be giving good seismic response by enhancing ductility.

B. Conclusions from Fragility Analysis

Probabilistic analysis is done to evaluate the damage statistics, and distinguish the buildings on the basis of their relative seismic performance. From the fragility analysis of different building using the capacity curves obtained from pushover analysis the following conclusions can be drawn:

- 1) The fragility curves indicate much higher damage probabilities for building designed with considering very low MCR value of 1.09.
- 2) The incorporation of higher MCR values reduces the damage probabilities irrespective of number of storey and damage level.
- 3) For 5-storey building increase of MCR beyond 1.26 does not decrease the probability of damage to an appreciable extent.
- 4) For 7-storey building wider variation of damage probability is observed from MCR 1.09 and 1.26 to MCR 1.47 for a given spectral displacement. From MCR 1.47 to 1.70 the probability of exceedance of a given damage state decrease but the difference is comparatively less. For MCR 1.7 to MCR 1.94 almost same damage probability is observed.
- 5) 10-storey building also shows same trend of fragility curves as of seven storey building frame.
- 6) So from the fragility analysis of the three type of buildings it can be concluded that, for up to 5 storey MCR 1.26 seems to be giving a lesser probability of damage. But as number of storey increases, i.e. up to 10-storey, MCR 1.47 may be appropriate to have good performance under strong earthquake shaking.

- 7) The MCR value depends on number of storeys and it may be higher when number of storeys increases.
- 8) A probabilistic framework gives complete insight into the expected performance of a building. So important for performance based seismic design.
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VII.FUTURE SCOPE

- 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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