A Study on the Adequacy of Codal Column-To-Beam Capacity Ratio

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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 preferred 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 plastic 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 variations among these codes and Indian standard is silent on this aspect. The present study aims at checking the adequacy of the codal provisions for C/B ratio in buildings. The C/B ratios are gradually increased and the failure of columns have been noted. It has been found through the present study that the codal provisions for C/'B ratio for buildings is not adequate, specially for tall buildings**

Keywords-- pushover, moment capacity ratio, fragility, ductility, lateral strength.

I. INTRODUCTION

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

$$Mc \;\geq\; Mb \qquad (1)$$

Where Mc and $Mb$are sum of the moment capacities at the end of column and beam meeting at a joint in a particular direction.

II. LITERATURE REVIEW

Jain *et. al.* (2006) proposed that, for a reinforced concrete moment resisting frame 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.

$$\sum Mc \;\geq\; 1.1\sum Mb(2)$$

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.

Many studies also have been conducted so far by the researchers in search of dominant collapse modes of the frames and designing strong column weak beam frames. Nakashima and Sawaizumi (1999) performed dynamic analysis taking ground motion as input in a fishbone shaped model and found that the required COF value that ensures beam hinging responses increases steadily with the increase in ground shaking. Medina and Krawinkler (2005) studied a family of regular frames to evaluate the strength demands suitable for the seismic design of the columns and indicated that the potential of formation of column plastic hinges is high for the frames designed as per the strong column weak beam requirements of current code provisions.

III. REVIEW OF CODES

Some international codes suggest expressions to prevent storey mechanism of collapse due to possible damage locations (hinge formations) in columns. This actually aims at achieving stronger columns with moment capacities more than those of beams framing into a joint obtained considering over strength factors. Moment Calculation at centre of the joint is a very complicated task. These moments are the design moment of resistance of columns or beams calculated at outer faces of the joint and a suitable allowance for moment obtained because of shear developed at the face of joint.

A. American Standard

ACI 318M-02suggests that "summation of moment capacities of column sections framing into a joint evaluated at the joint faces considering factored axial loads along the direction of lateral forces resulting in the minimum column moment, should be greater than or at least equal to 1.2 times the moment capacities of the beam sections framing into it.

$$\sum Mn,c \;\geq\; 1.2\sum Mn,b(3)$$

In equation (3), Mn,c and Mn,b represent moment capacities of columns and beams framing into a joint, calculated at joint face.

B. European Standard

EN1998-1:2003 recommends the following relation between moment capacities of columns to beams that is to be satisfied at all joints:

$$\sum Mn,c \geq 1.3\sum Mn,b \quad (4)$$

In equation (2.5) Mn,c is summation of the minimum moment capacity of the columns considering design axial forces and Mn,b is the summation of the moment capacities of the beams framing into the joint.

C. Indian Draft Standard

This issue of prevention of anchorage and shear failure in joint region during strong ground motions is not suitably addressed in the design and detailing recommendations for beam-column connections given in Indian standard. In view of these limitations, Jain *et al*. (2006) proposed a provision in draft code IS 13920:1993 for C/B ratio. According to that, in a moment resisting frame, designed for earthquake forces, at a joint summation of the moment capacities of the columns shall be at least equal to 1.1 times the summation of the moment capacities of the beams along each principal plane of the joint.

$$\sum Mn,c \geq 1.1\sum Mn,b \quad (5)$$

IV. METHODOLOGY USED

In order to study the optimum column beam capacity ratio, three RC frame buildings were taken, all having the same plan. The buildings were of different heights, the first one being of 5 storeys, second being of 10 storeys and the third being of 15 storeys. In order to maintain uniformity among all the models, some basic considerations were maintained.

1. The Reinforcement percentage in all the columns was kept at around 3-4%, and percentage steel in beam has been kept at 1-2% per face.
2. The original C/B ratio at all joints was maintained at not less than 1.3,
   i.e, C/B ratio ≥ 1.3.

After design, the buildings were subjected to a spectrum compatible ground motion which was obtained by as software developed by A Kumar (2004)**.** The background earthquake used was Kobe earthquake 1995. The magnitude of the earthquake was 7.2 on the Richter scale. The ground motion data was obtained from PEER website.

Using the spectrum compatible ground motion thus obtained, time history analysis was carried out on all the three buildings to look for possible failures of columns. The results obtained are discussed below.

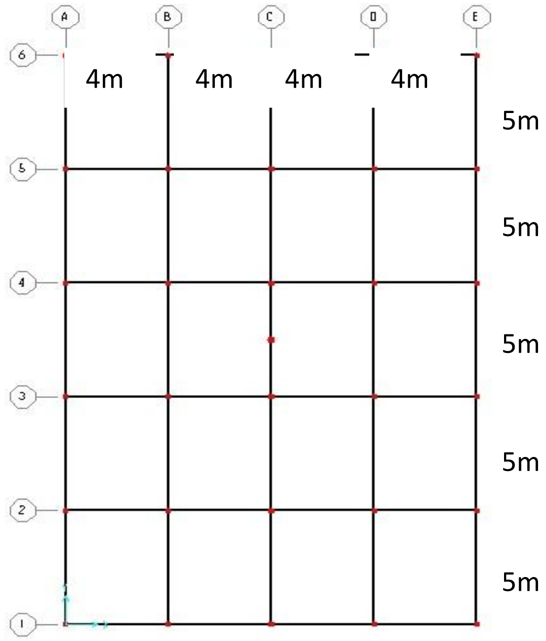

Fig 1: Building Plan considered

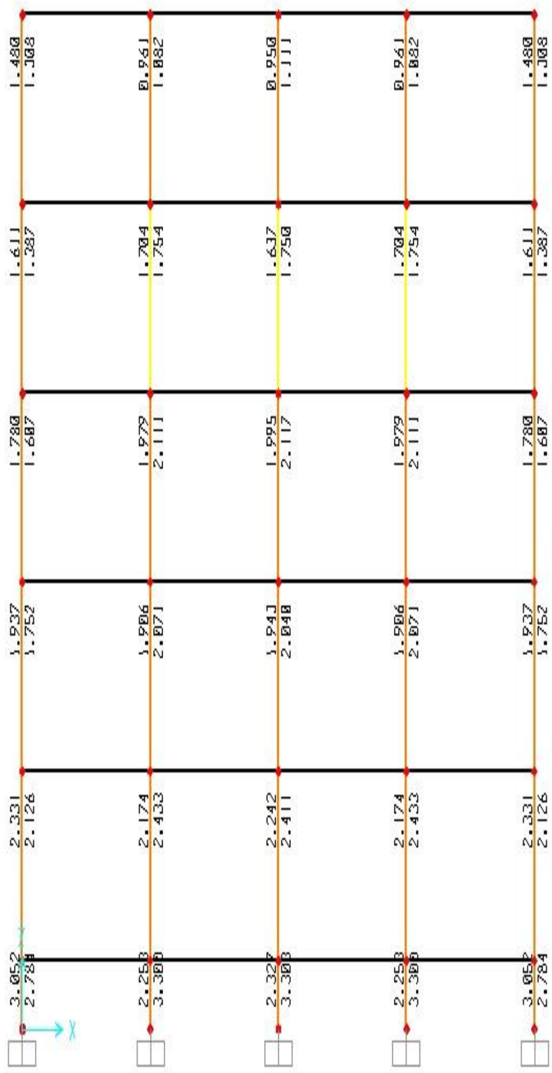

Fig 2: The C/B ratio after design in a typical frame

V. DISCUSSION ON THE RESULTS

1. There were no failures of columns in the 5-storey and 10-storey building when these were subjected to time history analysis.
2. There were failures in many columns through formation of plastic hinge for the 15-storey building under time history analysis.

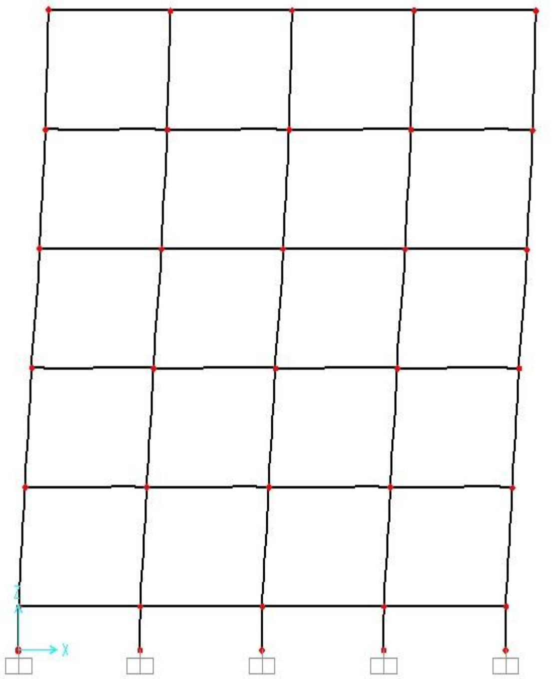

Fig 3: Failure of the columns of third building.

VI. CONCLUSION

From the results of the present study, it is found that the codalprovisions for C/B ratio for capacity design of buildings is not adequate for tall RC frame buildings. The 15-storey building showed formation of plastic hinges in many columns.

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