Proceedings of IOE Graduate Conference, 2017
Volume: 5 ISSN: 2350-8914 (Online), 2350-8906 (Print)

Effect of Column-to-Beam Moment Capacity Ratio on Seismic Performance of Reinforced Concrete Moment Resisting Frames

Sujan Bhandari [a], Hari Darshan Shrestha [b]

[a, b] *Department of Civil Engineering, Pulchowk Campus, Institute of Engineering, Tribhuvan University, Nepal*
Corresponding Email: [a] sujanbhandari2012@yahoo.com

Abstract

The damages caused by April 25th,2015 , Gorkha earthquake still remind us how vulnerable our cities are, like Kathmandu Valley. Though number of RCC building collapsed bears less percentage as a whole building damage, economic value of it is incomparable. Column sway mechanism was the main reason for most of the RCC building collapse. Though different code specify the moment capacity of column should be more than that of beam meeting at a joint by some percentage, our own National Building code is silent in this matter. Though adopting these strength enhancing parameters also doesn't seem to assure stopping column hinging. So urgency to study in this region was felt. 3-,5- and 8- storey intermediate RC frame of typical buildings are taken for study. The three set of structures are so designed that there are families of five structures in each set having different column to beam moment capacity ratio (CBMCR). Nonlinear static pushover analysis is done in SAP2000 for each structure to evaluate the effect of CBMCR in structure's lateral strength and displacement capacity. It is observed that increase in CBMCR increases lateral strength and displacement capacity of structures. Since there lie lots of variability within a structural damage, it is felt that probabilistic approach should be employed. So fragility curves are prepared for each set of structures to evaluate how fragile the structures are with varying CBMCR. Linear time history analysis is done in SAP2000 using Gorkha earthquake N-S component of accelogram. It is seen the structures with lesser CBMCR values have high probability of exceeding certain damage state than that of having higher values at same PGA. Another conclusion is made that these types of fragility curves can be assistance in design process for adopting suitable CBMCR value in joints of structure.

Keywords

CBMCR — Pushover Analysis — Time History Analysis — Fragility Curve

1. Introduction

It is well known fact that Nepal is located in seismically active region due to subduction of the Indian plate underneath the Eurasian plate. Many devastating earthquakes happened in the territory of Nepal in the past but most recently and most notably the Gorkha earthquake M7.8 of 25th april 2015 has shown that how much loss could an earthquake cause in vulnerable cities of ours like Kathmandu. The total damage and loss on Housing and Human Settlements is valued about US$ 3.5 billion which is about 50% of total nationwide loss caused by the very earthquake [1]. More than 7, 50,000 houses were collapsed or completely damaged in all affected districts and the number is more than 1, 15,000 within Kathmandu valley only.Though, percentage of failure of RCC building among total is less, the economic damage caused by it incomparable. Amongst various causes behind the failure of reinforced concrete buildings, storey mechanism,column sway or soft storey mechanism is a prime cause in such fatal failure. It is universally understood the fact that beam-column joint is one of the weakest components when a structure is subjected to lateral loading. Therefore such beam- column joint and its failure mechanisms need to be given prime attention.

The concept of capacity design attempts to set a strength hierarchy along the load path that aims to ensure that inelasticity is confined in some pre-determined and preferred structural components. 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. Capacity design procedure sets strength hierarchy first at the member level and then at the structural level. So, it needs adjustment of column strength to be more than the beams framing into it at a joint. Mathematically it can be expressed as

$$\Sigma M_c \geq \Sigma M_b$$

Where, M_c and M_b are moment capacities at the end of column and beam meeting at a joint respectively.

It is seen that in steel frames which dissipate energy in their columns may have poor seismic performance during major earthquakes [2]. Further the design concept of strong column weak girder usually implemented by some ratio of moment capacities of column to girder cannot actually prevent the occurrence of plastic hinges in columns [3]. Simple limit analysis can be done to demonstrate that a reasonable minimum column-girder strength ratio cannot be defined to eliminate yielding on columns of regular RC frames and suggested a strength reduction factor, Rg, to the girders in upper floor levels of the frame to reduce column yielding in the frames [4]. The research [5] proposed multi-objective seismic design method based on nonlinear static analysis and optimal column to beam strength ratio required for ensuring the beam hinge mechanism in both steel and RC moment resisting frames respectively for further design purpose. Another research [6] conducted series of nonlinear static and dynamic analysis in 9-story steel frames and concluded that deformation and the behavior of frames strongly depend on the relative strength distribution of the column bases and the beams. Moreover in research [7] non-linear analysis of 3-,9-,20- store moment resisting steel frames is done and the impacts of column beam strength ratio(CBSR) on member ductility demand, maximum inter-story drifts and floor acceleration amplification are investigated with varying CBSR. Fragility analysis was done to calculate design forces modification factors needed for achieving comparable probability of column yielding for different values of CBSR.

Some international codes suggest expressions to prevent storey mechanism of collapse due to possible damage locations (hinge formations) in columns by suggesting certain over strength factor of column with respective to beam meeting at a joint[8],[9],[10],[11],[12],[13].

Table 1: Overstrength Factor of Column over Beam Suggested by Different Codes

Code (Standard)	Overstrength Factor
USA	1.2
New Zealand	1.4
Europe	1.3
China	1.2 to 1.7
Bangladesh	1.2
India	1.4
Nepal	Not Available

2. Structural Modal Development

Intermediate frames of 3-,5- and 8- storey regular RC moment resisting buildings are selected and designed as per prevailing code[14],[15],[16]. Reinforcements in beams are adjusted in such a way that all the joints (except that of top stotey) have nearly equal CBMCR; but confirming to minimum and maximum limit of percentage of reinforcement specified by the code. Keeping beam reinforcement constant, column reinforcement is progressively increased to get five types of frames of similar storey with different representative value of CBMCR. Representative single value of each building frame is calculated by averaging CBMCR values of all joints, with some standard deviations. Each building frames are designed to have CBMCR values ranging from 1-2.2. Other design criteria are shown in tables 2.

Each building frame has beam section of 250mmX300 mm and column section of 350mmX350 mm. Storey height and bay width are taken as 3m and 3.5m.

3. Methodology

Pushover analysis of each building frame is performed in SAP2000 [17] to evaluate the lateral strength capacity, displacement capacity and ductility of structures. Default force deformation criteria for hinges

Table 2: Design Parameters

Zone	V
Exposure	Mild
Importance Factor	1
Soil Type	Medium
Concrete	$f_{ck} = 20MPa$ $Poisson'sratio = 0.18$ $Density = 25KN/m^2$ $ModulusofElasticity = 5000\sqrt{f_{ck}}$
Steel	Fe415
Dead Load	8.5 KN/m
Live Load	3.5 KN/m
Lateral Load	According to IS 1893(PartI):2002

Table 3: Building Frame Models and Respective Value of CBMCR

Building Frame	Average CBMCR	Standard Deviation
3-storey model-1	1.0	0.05
3-storey model-2	1.2	0.06
3-storey model-3	1.4	0.06
3-storey model-4	1.7	0.09
3-storey model-5	1.9	0.20
5-storey model-1	1.0	0.04
5-storey model-2	1.2	0.05
5-storey model-3	1.6	0.08
5-storey model-4	1.8	0.14
5-storey model-5	2.2	0.10
8-storey model-1	1.0	0.04
8-storey model-2	1.2	0.05
8-storey model-3	1.4	0.09
8-storey model-4	1.8	0.13
8-storey model-5	2.0	0.20

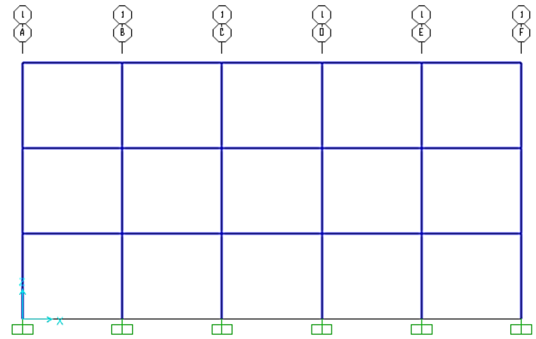


Figure 1: 3-Storey Building Frame

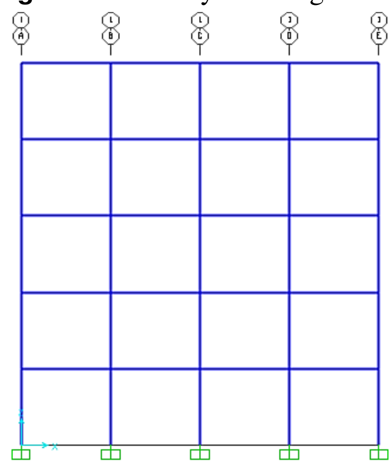


Figure 2: 5-Storey Building Frame

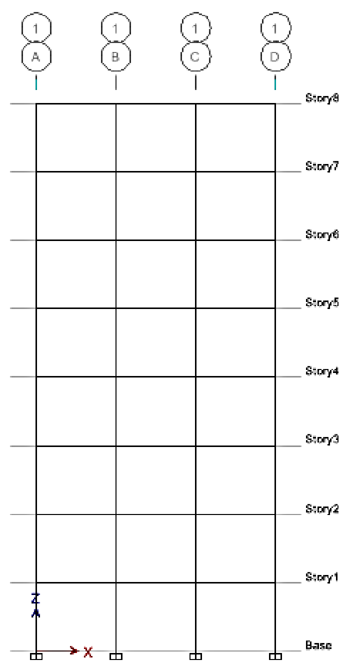
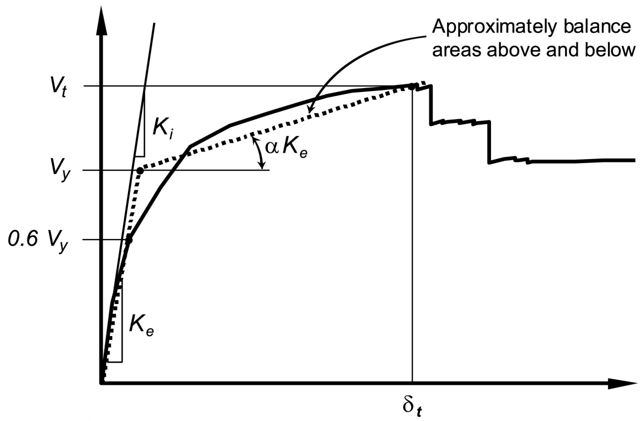
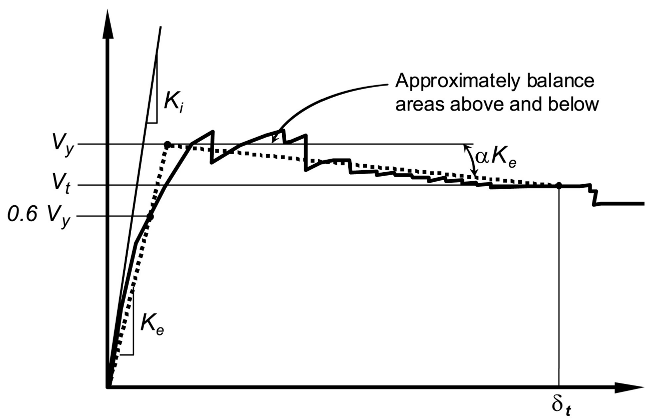


Figure 3: 8-Storey Building Frame



(a) Positive post-yield slope

Figure 4: Positive slope Equivalent Linearization



(b) Negative post-yield slope

Figure 5: Negative slope Equivalent Linearization

in beams and columns are used. The nonlinear force-displacement relationship between base shear and displacement of the control node shall be replaced with an idealized relationship to calculate the effective yield displacement and ultimate displacement of the building. This relationship shall be bilinear, with initial slope K_e and post-yield slope α.

Linear time history analysis of the structures is performed using SAP2000 for Gorkha earthquake N-S component of accelogram to find their peak displacement demands. Time history analysis was performed by direct integration method.

Fragility functions are developed to evaluate probability of exceeding a prescribed level of damage for a wide range of ground motion intensity. The levels of damages are categorized into four states i.e slight damage, moderate damage, extensive damage and

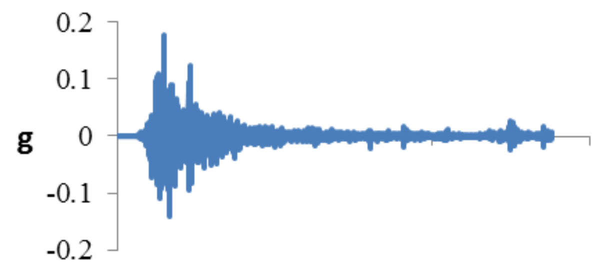


Figure 6: Gorkha Earthquake N-S Component

complete damage [19]. These damage states are numerically interpreted as a function of yield displacement (dy) and ultimate displacement (du) capacity whose values were calculated from pushover analysis as discussed in section 3.4. The four damage states (ds) were used as the capacity of the building in terms of yield displacement and ultimate displacement [20].

The spectral displacement demand, Sd is taken as the response of structure in terms of top displacement the structure for a given ground motion. The ground motion is input as time history data of the selected accelerograms. Then, the probability of exceeding a given damage state is modeled as a cumulative lognormal distribution. For structural damage, given the spectral displacement demand, Sd, the probability exceeding a damage state ds,is modelled as:

$$P[ds|S_d] = \phi\left[\frac{1}{\beta_{ds}}\ln\left(\frac{S_d}{S_{d,ds}{}^{-}}\right)\right]$$

Where: $S_d{}^{-}$, is the median value of spectral displacement at which the building reaches the threshold of the damage, ds

β_{ds} is the standard deviation of the natural logarithm of spectral displacement of damage state, ds, and

ϕ is the standard normal cumulative distribution function.

The total variability of each equivalent PGA structural damage state,$(SPGA)$ is modeled by the combination of following two contributors to damage variability:

• uncertainty in the damage-state threshold of the structural system $(\beta M(SPGA) = 0.4$ for all building types and damage states),

• Variability in response due to the spatial variability of ground motion demand $(V) = 0.5$ for long period spectral response).

The two contributors to damage state variability are assumed to be lognormally distributed, independent random variables and the total variability is simply the square-root-sum-of-the-squares combination of individual terms i.e ($\beta(SPGA) = 0.64$.

4. Results and Discussions

Pushover Curves: Force –deformation curve after pushover analysis for 3-,5- and 8- storey RCMRF are shown below. The curves indicate that lateral strength capacity and displacement capacity of structures increases with increase in CBMCR value.

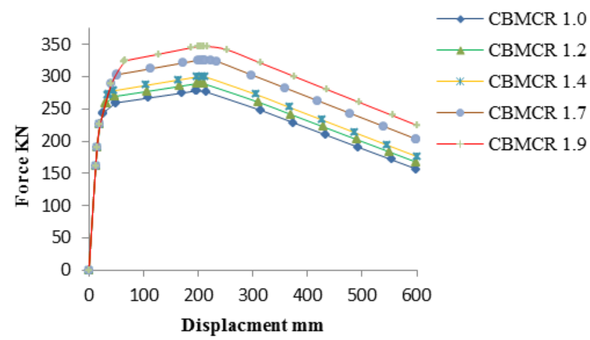


Figure 7: Pushover Curve for 3-Storey Frame

Displacement Ductility: Ratio of ultimate displacement capacity to yield displacement capacity is measure of structural displacement ductility, which is another major criteria for analyzing building performance. Global ductility of structures increases with increasing CBMCR value.

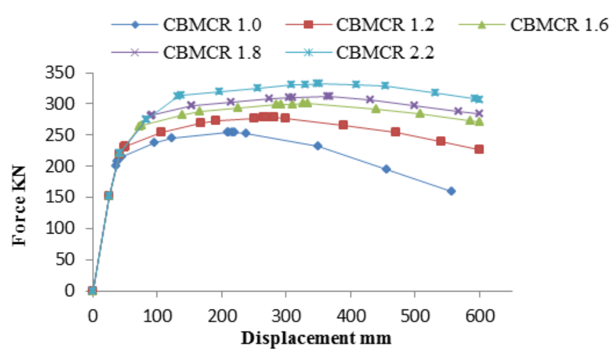


Figure 8: Pushover Curve for 5-Storey Frame

Fragility curves TFragility curves of structures with different CBMCR shows that increasing CBMCR reduces the probability of exceeding given damage state.

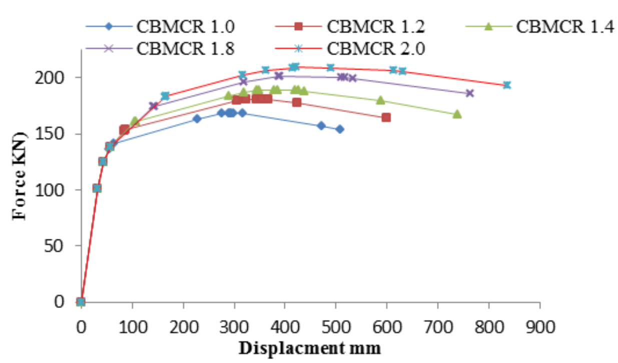


Figure 9: Pushover Curve for 8-Storey Frame

Table 4: CBMCR values for 3-storey Frame

Probability of Exceeding Damage state	CBMCR for PGA 0.36g					
slight	90%	92%	94%	96%	98%	100%
	>1.9	>1.9	>1.9	1.7−1.9	1.2−1.4	NA
Moderate	65%	70%	75%	80%	85%	90%
	>1.9	1.7−1.9	1.4−1.7	1.2−1.4	1.0−1.2	1.0−1.2
Extensive	5%	10%	15%	20%	25%	30%
	>1.9	1.7−1.9	1.2−1.4	1.0−1.2	1.0−1.2	1.0−1.2
Complete	1%	3%	5%	7%	9%	11%
	1.7−1.9	1.2−1.4	1.0−1.2	1.0−1.2	NA	NA

5. Conclusions

The main motive of this study is to come into the conclusion for suitable CBMCR of RC moment

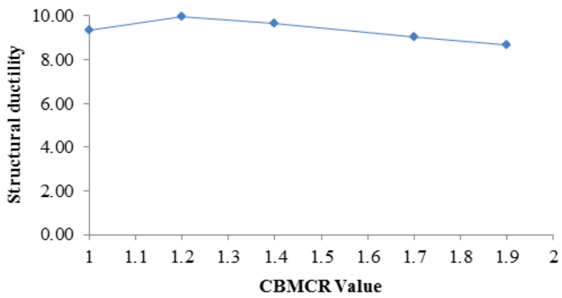


Figure 10: Displacement Ductility for 3-Storey Frame

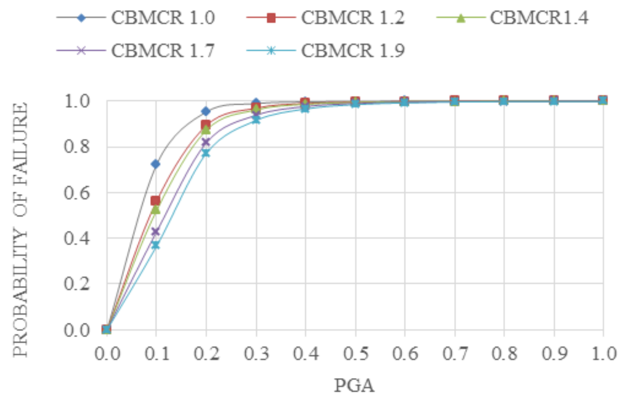


Figure 13: Fragility Curve for 3-storey Frame for Slight Damage State

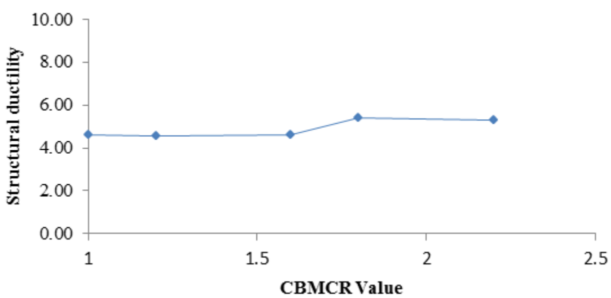


Figure 11: Displacement Ductility for 5-Storey Frame

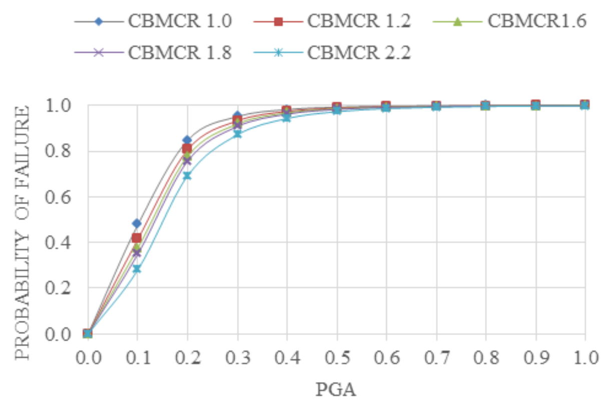


Figure 14: Fragility Curve for 5-storey Frame for Slight Damage State

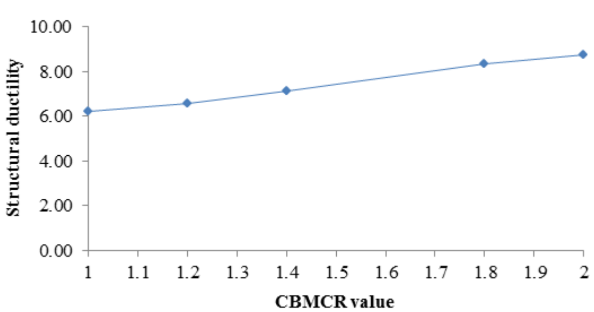


Figure 12: Displacement Ductility for 8-Storey Frame

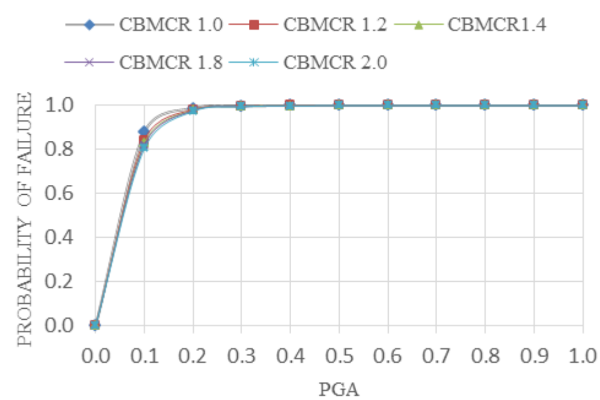


Figure 15: Fragility Curve for 8-storey Frame for Slight Damage State

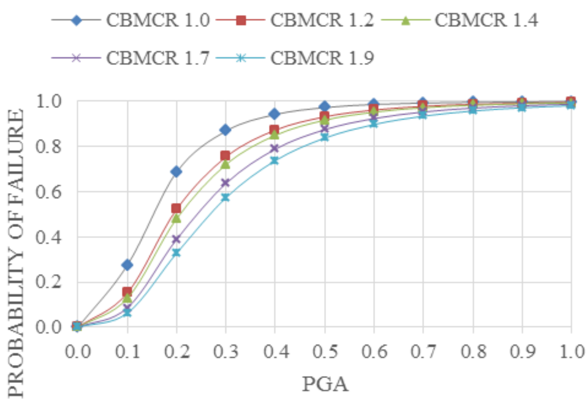


Figure 16: Fragility Curve for 3-storey Frame for Moderate Damage State

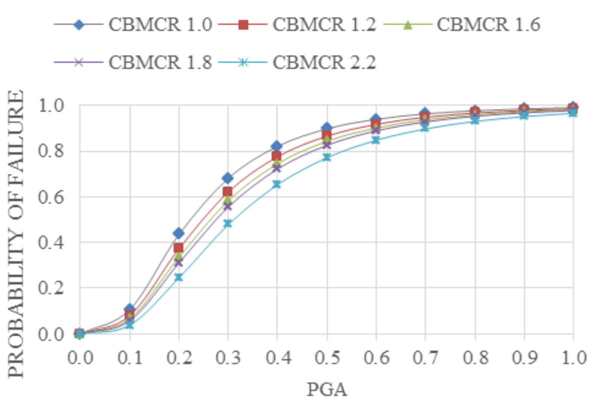


Figure 17: Fragility Curve for 5-storey Frame for Moderate Damage State

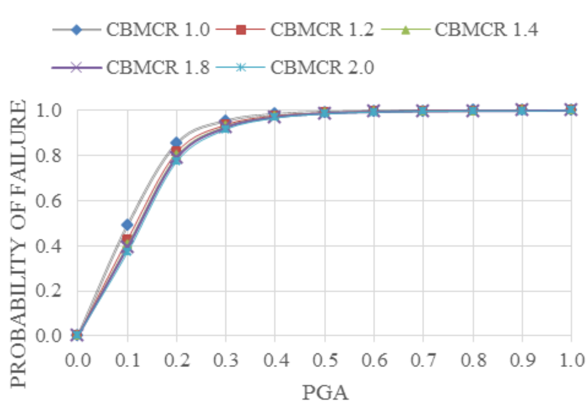


Figure 18: Fragility Curve for 8-storey Frame for Moderate Damage State

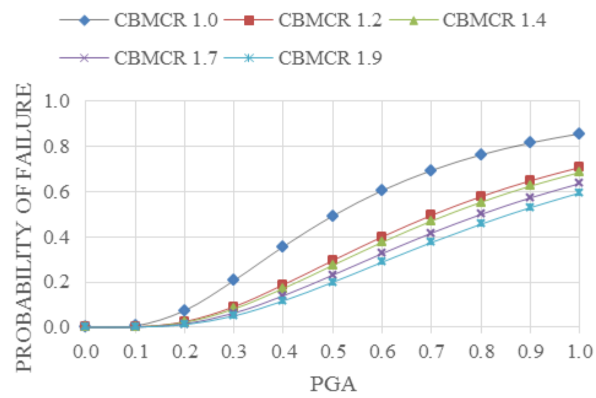


Figure 19: Fragility Curve for 3-storey Frame for Extensive Damage State

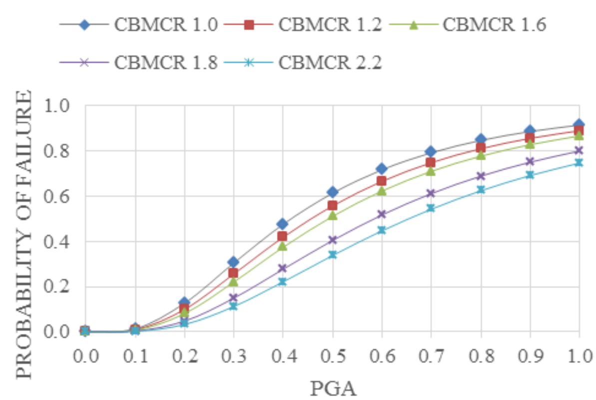


Figure 20: Fragility Curve for 5-storey Frame for Extensive Damage State

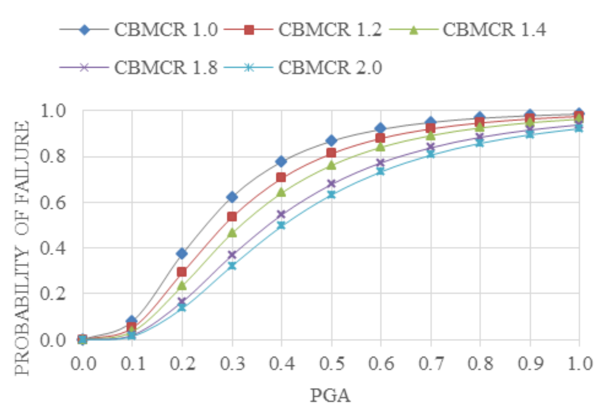


Figure 21: Fragility Curve for 8-storey Frame for Extensive Damage State

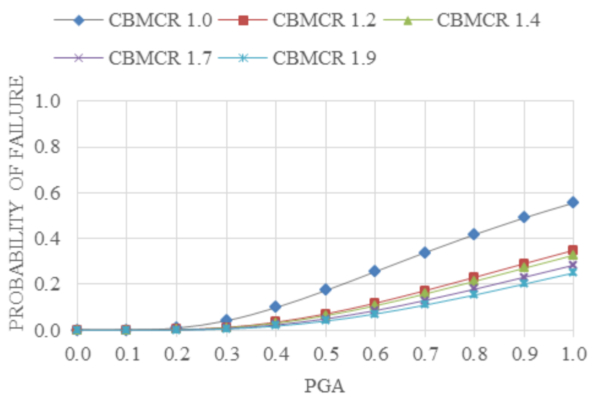


Figure 22: Fragility Curve for 3-storey Frame for Complete Damage State

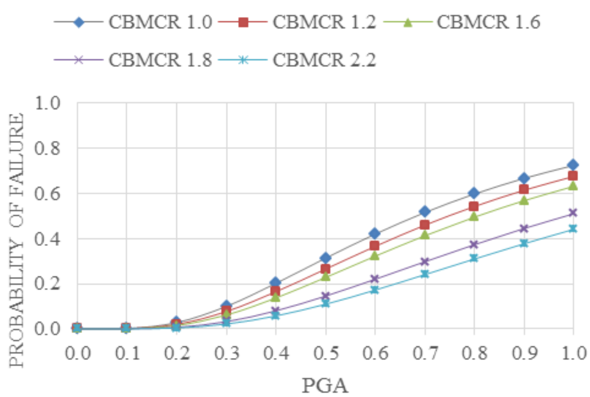


Figure 23: Fragility Curve for 5-storey Frame for Complete Damage State

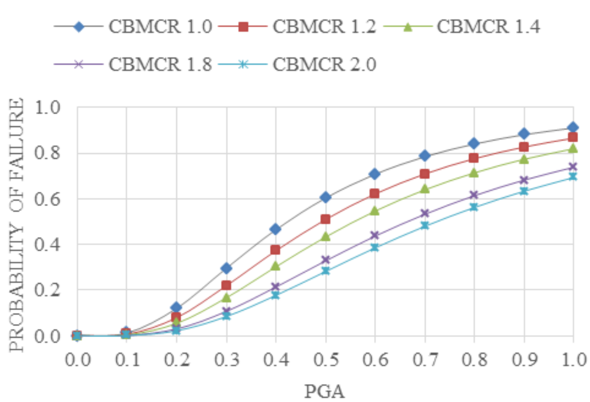


Figure 24: Fragility Curve for 8-storey Frame for Complete Damage State

resisting frames. Owing to different codes who adopt different CBMCR values, research is done on family of structures having different CBMCR ranging from 1.0 to 2.2. Following conclusions are drawn;

1. Pushover analysis of the family of structures shows that for a given storey RCMRF, irrespective of the CBMCR, all the structure follow same initial linear portion indicating CBMCR has nothing to do with elastic stiffness of structures.
2. Lateral yield displacement capacity of structures increases by 20-80% with increasing CBMCR from 1.0 to 2.0.
3. Lateral ultimate displacement capacity of structures increases by 60-70% with increasing CBMCR from 1.0 to 2.0.
4. Lateral strength capacity of structure increases by 25-30% with increasing CBMCR from 1.0 to 2.0.
5. Displacement ductility increases by 15-40% while increasing CBMCR value from 1.0 to 2.0.
6. Even by adopting CBMCR values adopted by different codes do not fully guarantee to stop column hinging thus restricting column sway failure of structure.
7. Probability of exceeding given damage state for same demand in structure decreases with increase in CBMCR of joints of the structure.
8. The fragility curves may be useful in design process of similar type of structures. Fixing the performance level of structure for given design earthquake, value or range of values of CBMCR to adopt may easily be found out by this method as shown in table 4.

6. Acknowledgements

I would like to express deep gratitude to my thesis supervisors Dr. Hari Darshan Shrestha for providing me valuable suggestions, helpful guidance, continuous encouragement and support throughout my thesis work. I would like to mention the name of my colleagues Er. Bibek Sigdel, Er. Sagar Karki and Er. Sunil Maharjan, who continuously helped me in many ways, throughout this work. How could I forget to remember my family?

Because without their support, I could not come this far. Lastly, I would like to thank all the staff at Department of Civil Engineering, Pulchowk Campus, IOE, who supported me throughout my research without hesitation.

7. References

[1] National Planning Commission. (2015). ”Nepal Earthquake 2015,Post Disaster Need Assessment Vol.:B Sector reports.” Kathmandu: Government of Nepal.

[2] Schneider, S. P., & W.Roeder, C. (1992). ”Behaviour of weak column strong beam steel frames.” Earthquake Engineering (pp. 2937-2942). Balkema: Rotterdam.

[3] Lee, H.-S. (1996). ”Revised rule for concept of strong column weak girder design.” Journal of Structural Engineering, 359-364.

[4] Kuntz, G. L., & Browning, J. (2003). ”Reduction of column yielding during earthquakes for reinforced concrete frames.” ACI Structural Journal, 573-580.

[5] Se Woon Choi, Y. K. (2013). ”Minimum column-to-beam strength ratios for beam–hinge mechanisms.” Journal of Constructional Steel Research, 53-62.

[6] Wongpakdee, N., & Leelataviwat, S. (2014). ”Effects of Column Capacity on the Seismic Behavior of Mid-rise Strong-Column-Weak-Beam Moment Frames.” European Conference on Earthquake Engineering and Seismology, (pp. 1-10). Istanbul.

[7] Arash E. Zaghi, S. S. (2014). ”Impact of column-to-beam strength ratio on the seismic response of steel MRFs”. Springer, 1-18.

[8] American Concrete Institute. (2008). ”Building Code Requirements for Structural Concrete and Commentary.” ACI 318M-08, 1-479.

[9] New Zealand Standard. (2006). ”Concrete Structures Standard.” NZS 3101:Part 1:2006, 1-309.

[10] National Standard of the People’s Republic of China.(2010).”Code for Seismic Design of Buildings.”GB 50011-2010.

[11] Bangladesh Standards (2006) . ”Bangladesh National Building Code.” BNBC.

[12] Bureau of Indian Standards. (2016). ”Ductile Detailing of Reinforced Concrete Structures Subjected to Seismic Forces - Code of Practice”. Draft Indian Standard IS 13920;2016, 1-37.

[13] GoN,MoPPW,DUDBC. (2003). ”Seismic Design Of Buildings.” NBC 105 : 1994, 1-34.

[14] Bureau of Indian Standards. (2000). ”Indian Standard Plain And Reinforced Concrete Code Of Practice.” IS. 456 : 2000, 1-114.

[15] Bureau of Indian Standards. (1980). ”Design Aid for Reinforced concrete to IS:456-2000.” SP-16:1980, 1-252.

[16] Bureau Of Indian Standards. (1986). ”Indian Standard Criteria For Earthquake Resistant Design Of Structures (Fourth Revision).” IS:1893-1984.

[17] FEMA . (2000). ”Prestandard And Commentary For The Seismic Rehabilitation Of Buildings.” FEMA 356, 1-518.

[18] Clough and Penzien (1993),Dynamics of Structures, Second Edition, McGraw-Hill, New York, ISBN 0-07-011394

[19] HAZUS-MH-MR3 (2003), ”Multi-hazard Loss Estimation Methodology, Technical Manual”, FEMA, Washington DC.

[20] Lagomarsino and Giovinazzi (2006).” Macroseismic and Mechanical Models for the Vulnerability and Damage Assessment of Current Buildings.” Springer Sceince+Business Media B.V, Vol. 4, pp 415-443