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Study on Literature Review of Strong Column Weak Beam Behavior of Frames

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Abstract*: Structural frames must have uniform energy distribution during seismic loading. Capacity design approach is used to achieve this, which demands "strong-column / weak-beam" to have good ductility and a desired collapse mechanism in the structure. When only the flexural strength of beams controls the overall response of a structure, RC beam-column link display ductile behavior .The failure mode having beams form hinges, is considered the most favorable mode for ensuring good global energy-dissipation. Therefore, it is necessary to study the Strong column-weak beam behavior of structures. In this paper, we reviewed the research carried out on this subject and published in many journals. It is observed from the study that the moment capacity ratio at beam column joint shall be more than one. Therefore, in this study we present different codal recommendations on achieving strong column weak beam behavior.

Keyword: Strong Column-Weak Beam, Pushover analysis, Moment Capacity Ratio/Ultimate moment ratio, Ductility, Plastic Hinges.

I. INTRODUCTION

During Earthquake, the structure is bond to experience vibration due to ground motions occurring in random fashions both horizontally and vertically, which induces inertia forces in them. Damage experienced by Moment Resisting RC framed structures, due to past earthquake show that failure may be due to utilization of concrete not having sufficient resistance, soft storey, beam-column joint failure due to improper anchorage and failure of column causing storey mechanism. When a structure is subjected to seismic loading, one of the potentially weaker components of the structure is beam-column connection. From past experiences, it can be noted that failure of structure is primarily at columns and beam-column joints. Hence study of seismic capacity of column or beam-column joint is topic of interest. Failure of beam is considered to be local failure whereas failure of column as global. Failure of beam can be easily renovated when compared to column as it is difficult to reconstruct as it affects the overall stability of the structure. In the following study a brief reviews over literatures available on Strong column-Weak Beam behaviour of structures is studied and presented.

II. LITERATURE REVIEW

Paper-1, "Analysis of Strong Column and Weak Beam Behaviour of Steel-concrete Mixed Frames" by Yangbing Liu, Yuanxin Liao and Nina Zheng. A large number of buildings destroyed in in Wenchuan earthquake on May 12th, 2008, shows column failure in RC Frame during the earthquake. One of the main reasons was insufficiently considering the cast-in-place floor slabs effect on the strength and stiffness of beams (Wang, 2008). According to the demand of seismic concept design of buildings, frame structures should have multiple lines of seismic resistance, one of which is the strong column-weak beam. It is achieved by improving the bearing capacity at column end near the node for RC frame structures and steel frame structures in Chinese code (GB50011-2010). Chinese code lacks methods to assure strong column-weak beam behaviour, as there is less research on this problem. In order to make the mixed frame structure system strong and sustainable under earthquake action, a practical method needs to be developed to realize strong column-weak beam behaviour of mixed frame structures.

In this paper a mixed frames consisting of concrete-filled steel tubular (CFT) columns and steel-concrete composite beams (CB) is considered for analysis. Pushover method is applied to study the failure mechanism of the CB-concrete-filled square steel tubular (CFST) column frame in this paper. The influence of ultimate moment ratio (column to beam) on the structural yielding mechanism is discussed.

A. Definition of Strong Column

Weak Beam As structural members lack strength reserve during an earthquake, the actual moment at beam and column end is equal to its flexural capacity. 'Strong column-weak beam' means that the actual flexural capacity of beam end M_b and Mc of column end at the node should meet the following inequality: Mc > Mb

B. Model Description

The two-span and three-story composite beam with full shear connection-CFST column plane frame is shown in Fig. 5. The length of beam span is L, and the height of the story is H. Accordingly, a lateral distributed loading with inverted triangle pattern based on first mode is adopted to perform the pushover analysis.

When conducting ultimate moment ratio analysis, the story height H of models is 3.6m; length of the section side of concrete-filled steel square tubular (CFST) column is 400mm; the section of composite beam is HN400×200; the width and thickness of RC flange is 1400mm and 100mm respectively.

For comparison's purpose, ratio of ultimate moment M_{cua} of CFST column under pure bending to that M'_{bua} of composite beam in the negative region, denoted by β_c, is used as ultimate moment ratio, shown in Eqn.

$\beta c = M_{cua} / M'_{bua}$

Different ultimate moment ratios are achieved by changing material strength, span and quantity of reinforcement in concrete flange plate of composite beam and material strength and wall thickness of steel tubular of CFST column. Pushover analysis is conducted when βc= 0.8, 1.0, 1.2, 1.6, 2.0 and 2.2 respectively. The standard value of dead load is 3.5kN/m, and the standard value of live load is 2.8 kN/m on the beams. The structure is applied one time dead load and live load before pushover analysis. And in the course of analysis, P-Δ effect is considered.

C. The influence of Ultimate Moment ratio (Strength Ratio)

Capacity curves are showed in Fig. 1.1. For different ultimate moment ratio βc. With the change of ultimate moment ratio from 2.2 to 0.8, the ductility of structures becomes poor. When $\beta c > 1.2$, ductility coefficient reduces slowly with the reduction of ultimate moment ratio within a certain range. When βc changes from 1.6 to 1.2, ductility coefficient rapidly decreases.

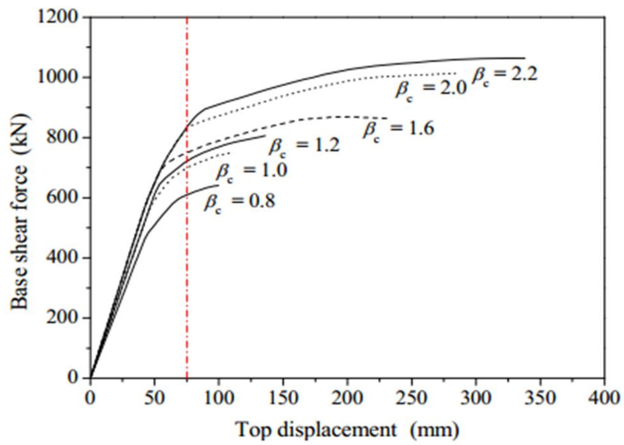

Fig. 1.1: Comparisons of capacity curves for different βc.

The position of the first plastic hinge, the damage state when the structural top displacement is 75mm (corresponding to the dotted line in Fig. 1.1.) and the ultimate failure mode of structures for different ultimate moment ratio are showed in Fig. 1.2 to Fig.1.4.

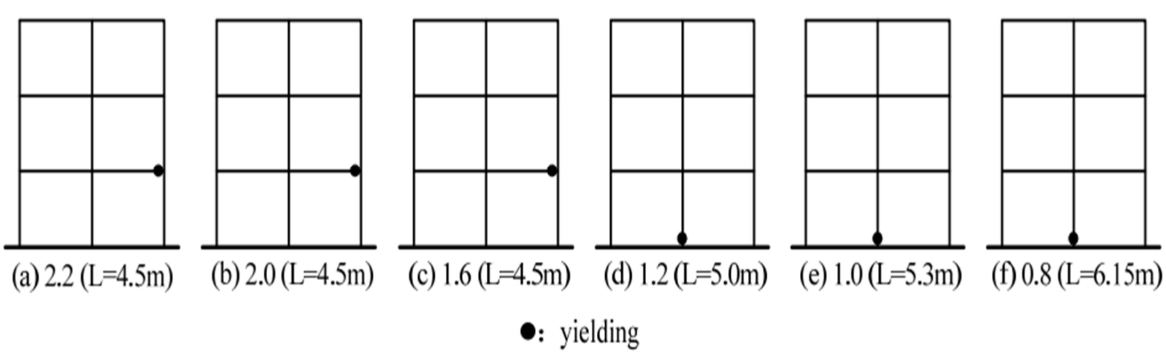

Figure 1.2. Location of the first plastic hinge for different βc

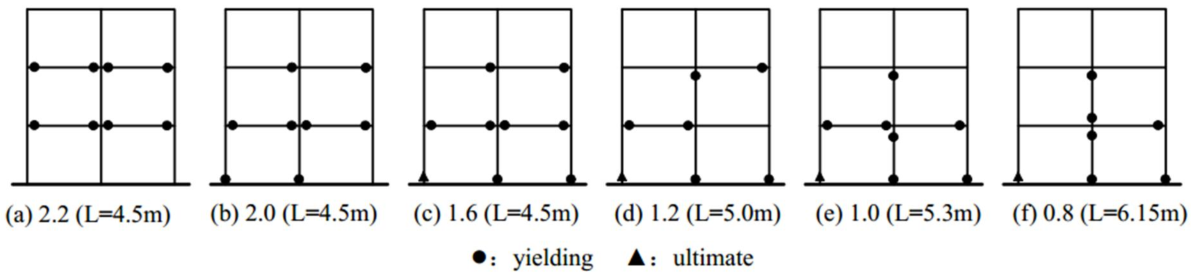

Fig. 1.3: Comparison of destruction state under top displacement 75mm for different βc

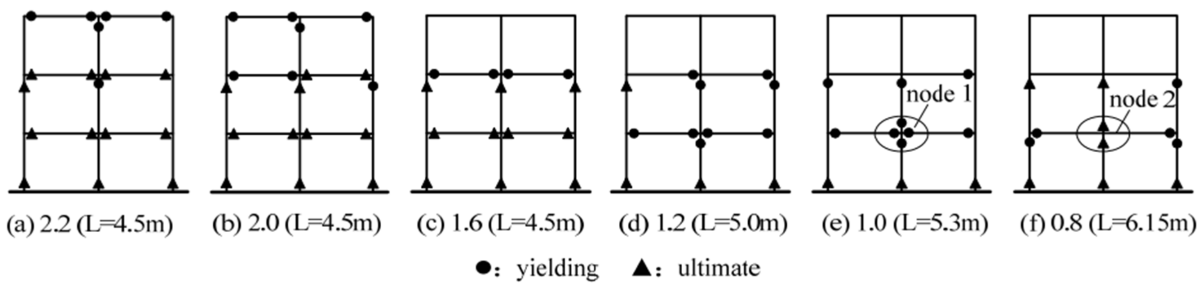

Fig. 1.4: Comparison of ultimate failure modes for different β

In contrast Fig.1.1 with Fig. 1.2-Fig. 1.4, as structure becomes more ductile, columns suffer heavy damage compared to beams with the change of βc from 2.2 to 0.8. For βc ≥1.6, Beam develops plastic hinge first, then the structure forms beam-hinge failure mechanism and finally the structure reaches ultimate state with the increased deformation and columns end failed in Fig. 4.2-Fig. 4.4. For βc ≥2, plastic hinges at beam-ends all develop well. For βc＝1.6, the bottom two storeys of the frame forms local failure mechanism and the structural ductility is not entire failure mechanism. For βc＝1.2, the plastic hinge firstly appears at the bottom of column, then at the beam end, and finally at the top of column. The frame is in the mixed failure state. For βc＝1, the plastic hinge firstly appears at the bottom of column too, and then at the beam end and the top of column almost at the same time. For βc＝0.8, the frame forms the column hinge mechanism. Corresponding to Fig. 1.1., the ductility of structures is very poor for βc＝1, 0.8.
From the above analysis, it cannot ensure the structure achieves strong column-weak beam yield mechanism. In order to obtain the strong column-weak beam yield mechanism, βc should take the value bigger than 1. βc =1.2 is the boundary value of the two yield mechanisms of the examples in this paper.

D. Conclusions

According to analysis results, design method to achieve strong column-weak beam for composite frames is preliminarily advised, which is applicable within certain axial compression ratio, and helpful for the other types of composite frames. But as the behaviour of composite member is largely different, further experimental research and theoretical analysis is needed for composite frames with different section compositions.

1) Paper-2, “Design recommendations for achieving ‘‘strong column-weak beam” in RC frames” by Ning Ning, Wenjun Qu and Zhongguo John Ma.: Reinforced Concrete (RC) frames are the most popular structural system for multistorey buildings in many parts of the world.
However, these buildings have shown poor performance during strong earthquakes in last few decades. For example, on October
8, 2005 an earthquake of 7.6 (Mw) struck the Kashmir of Pakistan, where the main damages of RC frames were the beam-column failure and the story failure (Fig. 2.1). Another earthquake of 6.2 (Mw) struck the Abbruzzo region of Italy on April 6, 2009. Seismic damage investigation showed that columns seem to have failed in compression before the yielding of beams (Fig. 2.2). Wenchuan China suffered a magnitude 8.0 earthquake on 12 May 2008. The main failure of RC frames was caused by ‘‘strong beam-weak column” (Fig. 2.3). Marmara earthquake of August 17, 1999 and Van earthquake on October 23, 2011 in Turkey showed that the slab affection was one of the reason why RC frames were damaged (Fig. 2.4). The similar failure

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modes were observed in the magnitude 6.6 earthquake in Bam on December 26, 2003 in Iran (Fig. 2.5) and in the magnitude of 7.6 earthquakes in Chi-Chi Taiwan in 1999. The "strong column-weak beam" concept was not implemented in the design of those school buildings. Thus, plastic hinges appeared in columns earlier than in beams (Fig. 2.6).

Fig. 2.1 Story failure of RC frames (Kashmir).

Fig. 2.4 Strong beam weak column failure (Turkey).

Fig. 2.2. Collapse of hotel (Italy).

Fig. 2.5. The plastic hinge in a weak column (Iran).

Fig. 2.3. Column damage (Wenchuan).

Fig. 2.6. Collapse of school (Taiwan).

The presented paper here first discusses the experiment of RC frames with cast in-situ slabs with the objective of investigating the participation of slabs during strong earthquakes. Then the FEA models considering different influence factors are analysed. The required ratio of column-to-beam strength is also proposed.

E. Experimental Investigation:

1) *Model description:* The experimental work consists of two 1:2.5 scaled spatial RC frames: a control specimen (RC-1) and a RC frame with cast in situ slabs (RC-2). Both frames are made of beams with a cross section of 100mm x 200mm and columns with a cross section of 160mm x 160mm. The thickness of RC-2 slabs is 50 mm. The frames were designed according to the Chinese

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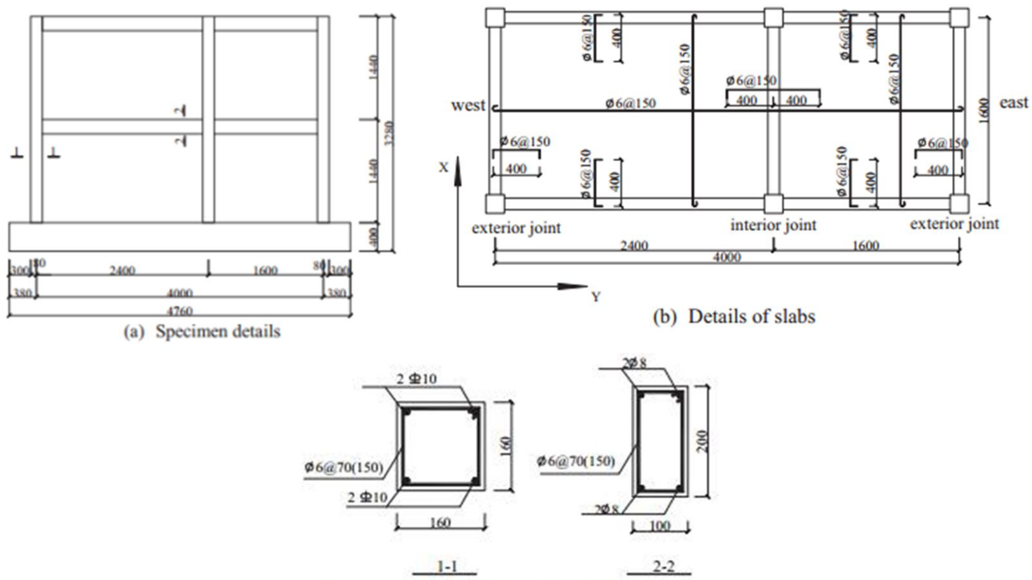
concrete structure code (GB50010-2008). Fig. 2.7 shows the geometry of the frames with two bays in the longitudinal (Y) direction and one bay in the transversal (X) direction.

2) *Test results:* The plastic hinges of the two frames are shown in Fig. 2.9. The failure pattern of RC-1 is the typical ‘‘strong-column-weak-beam”. For the RC-2, the failure pattern is the ‘‘strong-beam-weak-column” type which is different from the design objective of ‘‘strong-column-weak-beam”.

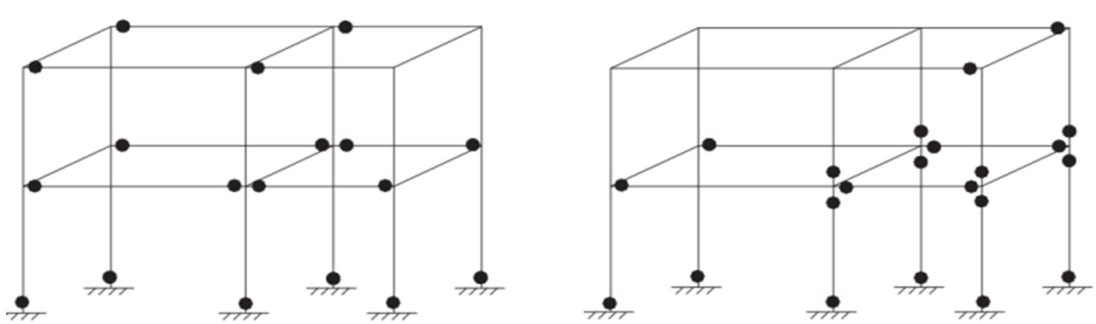


(a) Specimen details (b) Details of slabs

(c) Cross sections of column and beam

Fig. 2.7: Design of specimens.

Fig. 2.8: The loading setup.



(a) RC-1 (b) RC-2

Fig. 2.9: The location of plastic hinges.

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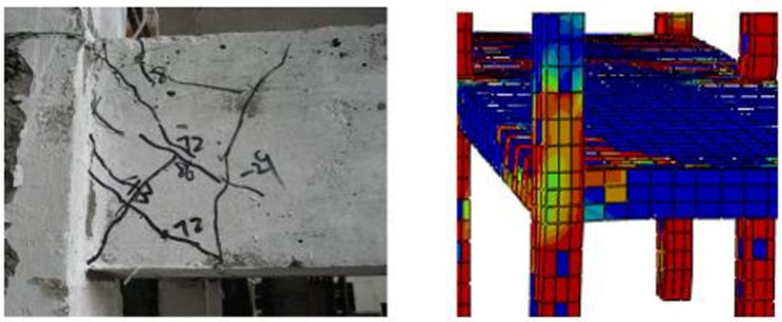
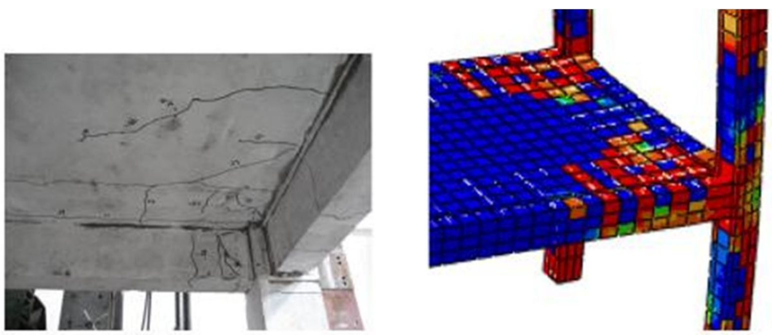
3) *FEM analysis:* Three dimensional spatial frame models have been developed using ABAQUS. Damaged plasticity model is used to simulate crack of the concrete. Reinforcement is embedded in concrete model. The main element size is 50 mm. The loading program is the same as the test one. The comparison of cracks from experiments with ‘‘simulation of damages” by FEA is shown in Fig. 2.10 and 2.11. The analysis results show that the FEA data fit the experimental results well. The axial compression ratio, concrete strength, reinforcement ratio of slabs, thickness of the slabs and the stiffness of the transverse beams are considered as variables in the FEA, as shown in Table 1. The actual axial load is calculated by scaled model in FEA models. The axial load ratios of RC-3 to RC-9 varies according to the proportions of each column in order to analyse the effective slab width and the failure patterns of RC frames while the axial ratio changes progressively. The stresses of the slab longitudinal reinforcement, beam longitudinal reinforcement and column longitudinal reinforcement are obtained from FEA results.



(a) test result (crack) (b) FEM result (damage)

Fig. 2.10: The torsion crack and damage of transverse beam



(a) test result (crack) (b) FEM result (damage)

Fig. 2.11: The crack and damage of slab

Table 1 Variables considered in FEA (The variables in different models are in bold type).

Models	Parameters							
	Axial compression ratio			Concrete strength/ MPa	Reinforcement ratio of slabs		Thickness of slabs/ mm	Dimension of transverse beam/mm
	East column	Middle column	West column		Reinforcement	Reinforcement ratio (%)		
RC-3	**0.10**	**0.18**	**0.13**	30	Φ6@150	0.377	50	100 × 200
RC-4	**0.20**	**0.36**	**0.26**	30	Φ6@150	0.377	50	100 × 200
RC-5	**0.25**	**0.45**	**0.33**	30	Φ6@150	0.377	50	100 × 200
RC-6	**0.30**	**0.54**	**0.39**	30	Φ6@150	0.377	50	100 × 200
RC-7	**0.35**	**0.63**	**0.46**	30	Φ6@150	0.377	50	100 × 200
RC-8	**0.40**	**0.72**	**0.52**	30	Φ6@150	0.377	50	100 × 200
RC-9	**0.50**	**0.90**	**0.65**	30	Φ6@150	0.377	50	100 × 200
RC-10	0.10	0.18	0.13	**20**	Φ6@150	0.377	50	100 × 200
RC-11	0.10	0.18	0.13	**25**	Φ6@150	0.377	50	100 × 200
RC-12	0.10	0.18	0.13	30	**Φ6@200**	**0.283**	50	100 × 200
RC-13	0.10	0.18	0.13	30	**Φ6@100**	**0.566**	50	100 × 200
RC-14	0.10	0.18	0.13	30	**Φ8@150**	**0.670**	50	100 × 200
RC-15	0.10	0.18	0.13	30	**Φ8@100**	**1.005**	50	100 × 200
RC-16	0.10	0.18	0.13	30	Φ6@150	0.377	**30**	100 × 200
RC-17	0.10	0.18	0.13	30	Φ6@150	0.377	**40**	100 × 200
RC-18	0.10	0.18	0.13	30	Φ6@150	0.377	**60**	100 × 200
RC-19	0.10	0.18	0.13	30	Φ6@150	0.377	50	**100 × 100**
RC-20	0.10	0.18	0.13	30	Φ6@150	0.377	50	**100 × 150**
RC-21	0.10	0.18	0.13	30	Φ6@150	0.377	50	**100 × 250**
RC-22	0.10	0.18	0.13	30	Φ6@150	0.377	50	**100 × 300**

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Table 2 Ratios of column-to-beam strength at the 2% story drift.

Models	East exterior joint			Interior joint			West exterior joint		
	$\sum M_c$/kNm	$\sum M_b$/kNm	$\eta\ (\sum M_c/\sum M_b)$	$\sum M_c$/kNm	$\sum M_b$/kNm	$\eta\ (\sum M_c/\sum M_b)$	$\sum M_c$/kNm	$\sum M_b$/kNm	$\eta\ (\sum M_c/\sum M_b)$
RC-2	8.74	13.09	0.67	13.34	14.27	0.91	9.47	13.32	0.71
RC-3	11.22	15.55	0.72	11.03	16.43	0.67	8.74	15.48	0.56
RC-4	13.25	15.00	0.88	34.30	15.78	2.17	23.90	12.17	1.96
RC-5	20.95	15.20	1.38	43.40	16.31	2.66	30.86	12.95	2.38
RC-6	32.93	15.92	2.07	56.49	17.56	3.22	43.81	14.93	2.93
RC-7	34.85	16.28	2.14	59.04	18.00	3.28	48.70	15.62	3.12
RC-10	9.39	13.62	0.67	13.23	15.93	0.83	7.56	13.67	0.55
RC-11	9.66	14.74	0.66	13.31	15.54	0.86	10.69	13.74	0.78
RC-12	8.56	13.99	0.61	13.31	16.30	0.82	10.31	14.82	0.70
RC-13	9.64	17.15	0.56	13.31	14.22	0.94	11.15	17.85	0.60
RC-14	10.68	17.76	0.60	13.31	15.56	0.86	9.80	16.51	0.59
RC-15	10.73	19.60	0.55	13.31	17.89	0.74	10.03	19.94	0.50
RC-16	10.83	15.79	0.69	13.31	16.66	0.80	10.35	15.10	0.69
RC-17	11.02	16.33	0.67	13.31	16.18	0.82	8.45	15.59	0.54
RC-18	12.53	16.04	0.78	13.31	17.33	0.77	10.54	14.86	0.71
RC-19	7.29	13.54	0.54	7.29	11.22	0.65	7.29	12.83	0.57
RC-20	7.51	13.99	0.54	9.26	12.98	0.71	7.91	13.99	0.57
RC-21	8.12	16.35	0.49	11.65	14.98	0.78	8.64	15.02	0.58
RC-22	8.86	17.31	0.51	13.19	17.28	0.76	9.03	16.33	0.59

4) *The required ratio of column-to-beam strength:* The required ratio of column-to-beam strength can be defined as Eq.

$$\eta = \frac{\sum M_c}{\sum M_b}$$

In which, Mc = sum of moment of columns calculated at the 2% story drift; Mb = sum of moment of beams calculated at the 2% story drift. Where the slab is in tension under moments at the face of the joint, slab reinforcement within an effective slab width can be calculated with the proposed equations above. Table 2 shows the calculated results of the required ratio of column-to-beam strength at the 2% story drift based on FEA analyses. It can be concluded that the required ratio of column-to-beam strength η, is increased when the axial compression ratio is increased. When the axial compression ratio reaches 0.39, η is about 3.0. In order to ensure a ‘‘strong column weak beam” failure mode, Eq. (14) should be used.

$$\sum M_c \geqslant \eta \sum M_b$$

Where η = 1.5, 2.5, 3.0 when the axial compression ratio of column is smaller than 0.25, within the range of 0.25–0.4, and larger than 0.4, respectively.

5) *Conclusions:* Based on the experimental investigation and FEM analyses, the following conclusions can be drawn:

1) Experimental results demonstrate that slabs can change the failure pattern of RC frames from a typical ‘‘strong column weak beam” failure to the ‘‘strong beam weak column” failure.
2) The axial compression ratio is the most sensitive factor influencing the required ratio of column-to-beam strength. This ratio increases with the increase of the axial compression ratio. When the axial compression ratio reaches 0.39, this ratio reaches 3.0. The reasonable ratio of column-to-beam strength is proposed to avoid the brittle failure of the RC Frames in seismic design.

III. REVIEW OF CODES

Some international codes recommend expressions to obviate storey mechanism of collapse due to potential damage locations (hinge formations) in columns. The aim of this is actually to attain stronger columns with moment capacities higher than those of beams framing into a joint obtained, considering over strength factors.

A. American Standard

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

$$\sum M_{nC} = 1.2 \sum M_{nB} \quad \text{…… (3.1)}$$

In equation (3.1), moment capacities of columns and beams framing into a joint are represented by M_{nC} and M_{nB}.

IV. NEW ZEALAND STANDARD

The capacity design philosophy needs for the design that of column, under flexure moment of resistance of columns must be greater than the moment of resistance of beams framing into a joint including the over strength of the beams. New Zealand Standard (NZS3101:1995) suggests this aspect with respect to center of the joint as follows:

$$\sum M_c \geq 1.4 \sum \alpha \, M_b \quad \text{(3.2)}$$

In equation (3.2) α is over strength factor for beams. The over strength of steel reinforcement is considered as 1.25 and strength reduction factor is taken as 0.85. Hence the over strength factor to be considered for beams is 1.47.

V. EUROPEAN STANDARD

EN1998-1:2003 recommends the relation between moment capacities of columns to beams at all joints:

$$\sum M_{nc} = 1.3 \sum M_{nb} \quad \text{(3.3)}$$

In equation (3.3) Mnc is summation of the minimum moment capacities of the columns considering design axial forces and Mnb is summation of the moment capacities of the beams framing into the joint.

VI. INDIAN STANDARD

In the design and detailing recommendations for beam column connections given in Indian standard this issue of prevention of anchorage and shear failure in joint region during strong ground motions is not addressed properly. Jain et.al. (2006) proposed a provision in draft for inclusion in IS13920:1993, According to which, in a moment resisting frame designed for earthquake forces, at the 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.

According to the revised code of IS 13920:2016, this aspect of moment capacity ratio is considered as At a beam-column connection of a framed structure, the total summation of nominal flexural strength of columns meeting that joint along each principal plane shall be at least 1.4 times the summation of flexural strength of beams meeting at that joint in that same plane.

Mathematically it is expressed as in equation (3.4)

$$\sum M_{nc} = 1.4 \sum M_{nb} \quad \text{(3.4)}$$

VII. CONCLUSIONS

A. With the change of ultimate moment ratio from 2.2 to 0.8, the ductility of structures becomes poor with reduction in ultimate moment ratio. [Paper-1]

B. According to analysis results, the method by adjusting the elastic inner force for RC frame cannot give a guarantee to achieve the strong column-weak beam yield mechanism. [Paper-1]

C. Based on Experimental and FEM analysis, Slab can change the failure pattern of structure because it contributes in resisting moments from Strong Column-Weak Beam to Weak Column-Strong Beam. [Paper-2]

D. Based on codal reviews, there are lot of discrepancies amongst these international codes to achieve strong column weak beam behavior. Therefore it's important to study Strong Column-Weak Beam behavior of structures.

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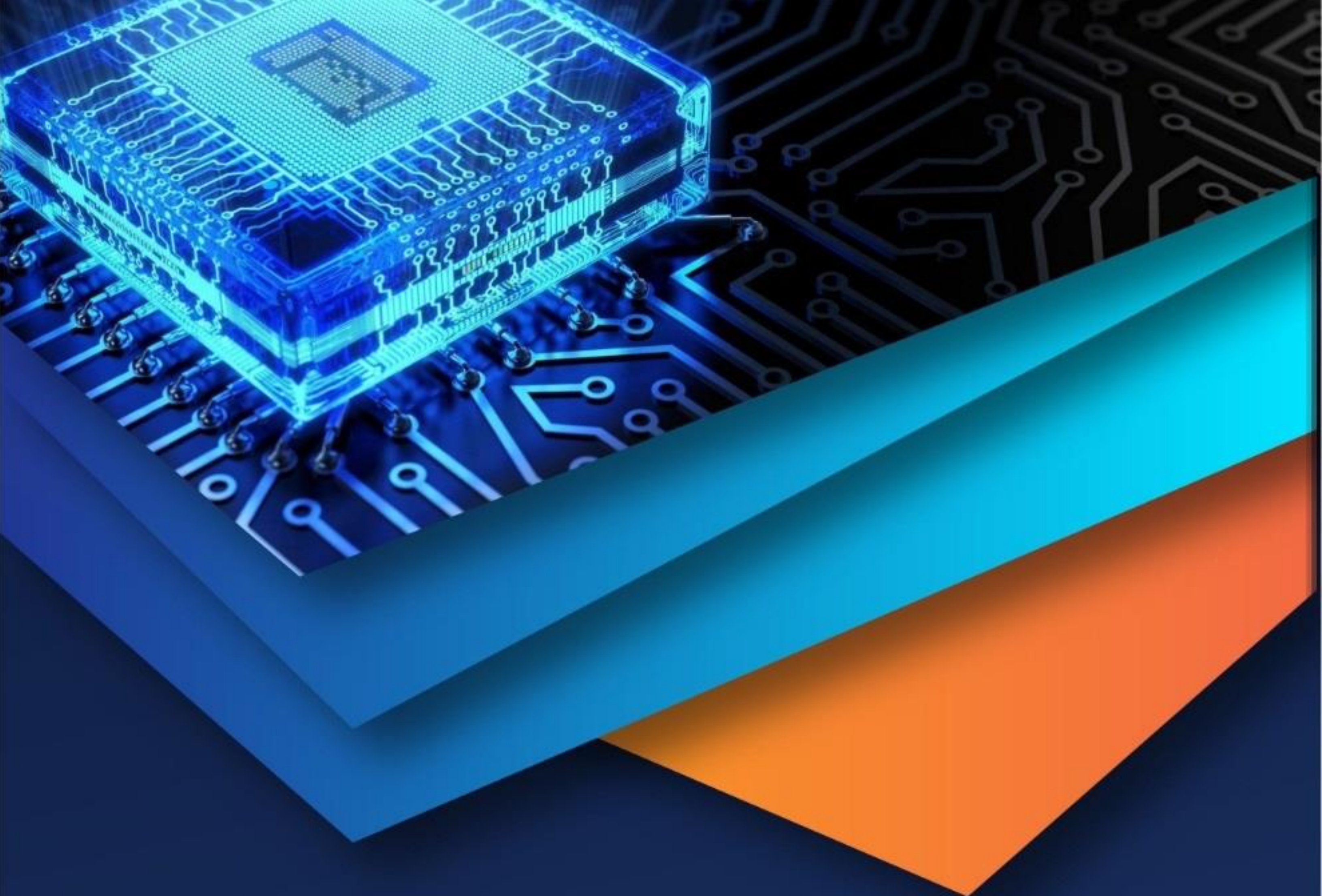

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