# Effects of Column-to-Beam Strength Ratio on Behaviour of Beam-to-Column Moment Joints

Arnav Anuj Kasar, S. D. Bharti, M. K. Shrimali and Rupen Goswami

**Abstract** Steel Moment Resisting Frames (MRFs) are believed to be very ductile structures and are highly regarded structural systems, even in areas of high seismicity. The overall ductility of a steel MRF depends on many factors, such as connection configuration, column-to-beam strength ratio, effect of joint panel zone, material and cross-sectional properties, rate of loading, etc. Seismic performance of a steel MRF depends, primarily, on the performance of beam-to-column joints in the frame. The capacity of a joint to undergo inelastic deformation determines the ductility of an MRF. In a beam-to-column moment joint, the desirable behaviour is to limit inelastic actions at beam end regions, thereby preventing irreparable damage to structures. In a simple unreinforced moment connection, this is achieved by varying the Column-to-Beam Strength Ratio (CBSR). In this paper, a minimum value of CBSR, which ensures the formation of plastic hinge at beam end region, for a simple unreinforced joint, is determined. A parametric study, using Nonlinear Finite Element Analysis (NFEA), is carried out to determine the force–deformation behaviour of ten beams to column joint subassemblages. The CBSRs are varied from 1.2 to 11 to determine the value at which inelastic actions can be limited to the beam ends. The selection of CBSRs is based on the strength of AISC standard sections and their compatibility along with the various prevalent codal provisions. Results of NFEA show that the minimum CBSR required to prevent inelastic actions in columns is close to 7.5.

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Capacity design · Strong axis moment joints · Force–deformation behaviour
Strong column weak beam

## 1 Introduction

Since their inception, steel Moment-Resistant Frames (MRFs) are considered to be the most effective structural systems. This belief is based, partially, on the inherent ductility of steel as a construction material, and partially on their ease of construction and architectural suitability. During mid-twentieth century, steel MRFs were the most preferred structural systems, especially in areas of high seismicity. It was because of their 'assumed' good ductile behaviour that these structural systems were assigned the most optimistic design parameters.

The damages sustained by steel MRFs during and in the aftermath of 1994 Northridge and 1995 Kobe earthquakes were phenomenal, and led to substantial research on the behaviour of such frames, under seismic excitations [1]. The Northridge earthquake led to the damage of nearly 200 steel MRF buildings, most of which were located in the connection region. The damage to welded beam-to-column connections can be attributed to two broad reasons: (i) improper design, leading to insufficient strength of connections; and (ii) excessive rotation, due to inelastic yielding of Joint Panel Zone (JPZ) region. As MRFs are more flexible than other common earthquake-resistant structural systems, such as braced frames and steel plate shear wall systems, they require to be designed on the basis of drift limits. When subjected to lateral forces, the lateral deformation of steel moment frames is supposed to be accommodated through inelastic flexing of beams, while the columns are supposed to remain in elastic range.

The concept of capacity design recommends that, in a frame structural members, should be proportioned to yield at predetermined sequence and locations. Thus, better control on the inelastic behaviour of structures is achieved, by accommodating the imposed ductility demand in a few predetermined locations, while rest of the structures remains in elastic range. A typical strong axis, interior beam-to-column joint subassemblage, of an MRF, is shown in Fig. 1, depicting different regions of the subassemblage. The concept of capacity design suggests that both the columns and the beam-to-column joints have to be stronger than the beams [2].

In the past, numerous researches have been carried to determine the factors on which the ductility of steel beam-to-column moment joint depends. One of the most critical factors is design of Joint Panel Zone (JPZ) region, which may be based on one of the three design philosophies: (i) Strong JPZ, wherein the JPZ remains elastic [3], forcing all inelastic actions to the beam ends; (ii) Weak JPZ, all inelastic deformations are limited to the JPZ [4] region, thereby preventing the formation of plastic hinges at beam ends; and (iii) Balanced JPZ, allows controlled inelastic yielding of JPZ [5–7] region, leading to sharing of inelasticity between beams and

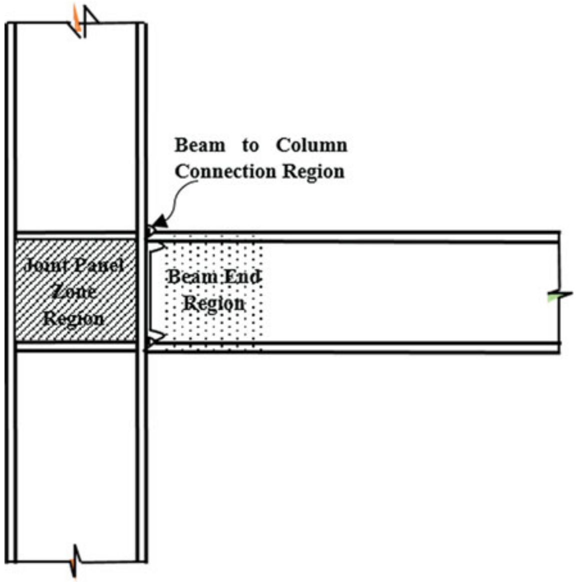


**Fig. 1** A typical exterior beam-to-column moment joint subassemblage showing different regions of the joint

JPZ region. Most of the prevalent design codes recommend that the JPZ region shall be designed on the basis of the third approach, i.e., sharing of inelastic actions between beams and JPZ region.

When subjected to lateral force, JPZ region undergoes very stable shear yielding and is capable of undergoing large inelastic deformations [5]. To utilize this reserve strength, designers configured the joints such that the JPZ region is allowed to participate, along with the beams, in dissipating the input seismic energy through inelastic action. Much research has been carried out to determine the effects of behaviour of JPZ region on the overall performance of MRFs [8–12].

The objectives of code-based design are to assure life safety (strength and ductility) and, to some extent, prevent damage (limiting the drifts). The first objective may be achieved by allowing yielding of JPZ region; nevertheless, the shear yielding of JPZ region leads to uncontrolled overall deformation of the structure. Also, as the JPZ is an integral part of column, shear yielding of JPZ necessarily means yielding of column web and is irreparable in nature, and thus shall not be permitted. In the prevalent state of steel design, the JPZs are designed to undergo yielding simultaneously with the beams. This design philosophy needs to be revisited and suitable amendments need to be brought in, so that the shear yielding of JPZ can be postponed up to the formation of beam plastic hinge.

## 2 Modelling and Analysis

To determine the effects of CBSR on the behaviour of strong axis interior beam-to-column joint subassemblages, a range combination of columns and beams is selected. Two important deciding parameters for selecting a section as column or beam are (a) plastic section moduli and (b) width of flange. Beam sections are such selected, in which the width of beam flanges remains lesser than width of corresponding column flanges for a particular CBSR. A fair representation of a wide range of column-to-beam strength ratios is achieved through section selection. Ten beam-to-column joint subassemblages are modelled having different CBSRs, ranging from 1.2 to 11. A list of different beam and column sections, used to obtain beam-to-column joint subassemblages with different CBSRs, is presented in Table 1. The class of selected section is determined using tables B4.1a and B4.1b of AISC 361-10, and most of the sections selected are compact, while only a few are classified as non-compact.

For the purpose of analysis, both exterior and interior beam-to-column joint subassemblages are modelled starting from CBSR of 1.2, based on IS 800:2007. The subassemblages consist of column with height equal to sum of the distance of point of contraflexure, above and below the joint. Beam length for subassemblage is also taken to be equal to distance between two points of contraflexures, on either side of the column for an interior joint. The points of contraflexures are assumed at the mid-heights of members, and centerline dimensions are considered at this stage (Fig. 2).

The subassemblages are simply supported at column ends, and displacement loading is applied at beam ends. A displacement-based nonlinear finite element analysis of all the subassemblages is performed using ABAQUS software package [13]. The members are assumed to be of ASTM A36 grade steel with isotropic hardening model, the stress–strain behaviour of which is shown in Fig. 3 (yield stress of 250 MPa and ultimate stress of 415 MPa).

**Table 1** Standard AISC sections used for beam-to-column joint subassemblages

| S.N. | Beam sections | | Column sections | | CBSR |
|---|---|---|---|---|---|
| | Section | $M_{pB}$ (kNm) | Section | $M_{pC}$ (kNm) | $M_{pC}/M_{pB}$ |
| 1. | W27×84 | 1,000 | W18×130 | 1,188 | 1.19 |
| 2. | W18×97 | 864 | W24×176 | 2,093 | 2.42 |
| 3. | W18×71 | 598 | W24×176 | 2,093 | 3.50 |
| 4. | W24×103 | 1,147 | W33×318 | 5,203 | 4.54 |
| 5. | W30×124 | 1,671 | W36×529 | 9,545 | 5.71 |
| 6. | W27×102 | 1,250 | W36×487 | 8,726 | 6.98 |
| 7. | W21×101 | 1,036 | W27×539 | 7,743 | 7.47 |
| 8. | W30×90 | 1,159 | W40×503 | 9,504 | 8.20 |
| 9. | W27×84 | 1,000 | W36×529 | 9,545 | 9.55 |
| 10. | W24×94 | 1,041 | W40×593 | 11,307 | 10.86 |

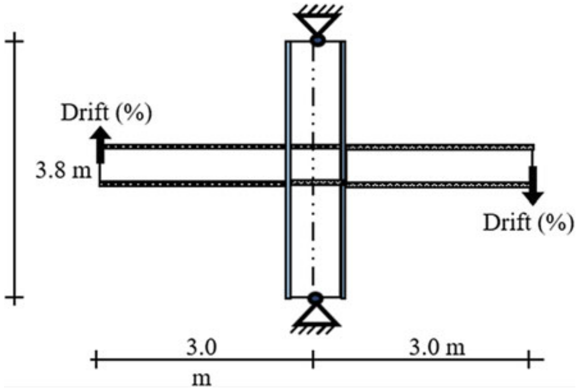


**Fig. 2** Strong axis interior beam-to-column joint subassemblage modelled for analyses

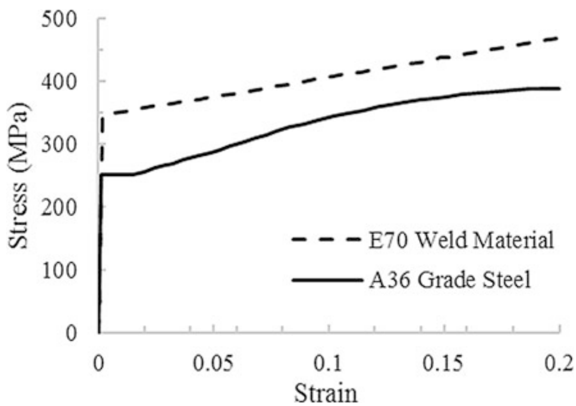


**Fig. 3** Stress–strain relationship of materials used

Stress–strain relationships for A36 grade steel and E70 electrodes used for analysis are shown in Fig. 3. The modulus of elasticity and Poisson's ratio of both the materials are 200 GPa and 0.260, respectively. The height of columns in the subassemblages is 3.8 m, which, in most cases, is the average storey height (Fig. 2). The distance considered between column centerline and the point of application of load on beams is 3.0 m, representing span of beam. Nonlinear analyses are carried out on three-dimensional solid models. A uniform mesh is developed for the subassemblage models using eight-noded linear brick element (C3D8R). Single-step monotonic drift loading up to a drift level of 4% is used for analyses to obtain the differences in responses of these beam–column joints. Axial compressive load is not considered on the columns, to reduce the number of parameters on which the behaviour of subassemblage depends.

## 3 Results and Discussion

Displacement-based nonlinear finite element analyses are carried out for ten beam-to-column joint subassemblages. The results obtained from ABAQUS are presented in Figs. 4 and 5. Figure 4 shows the level of inelasticity induced in a beam-to-column joint, at a drift level of 4% through von-Mises stress contours. For combinations having a CBSR less than 7, inelastic yielding of JPZ is observed. The extent of inelasticity in JPZ region reduces, with an increase in CBSR. This is primarily due to increase in strength of beams, which in turn increases their participation in the dissipation of induced energy, through inelastic excursions.

Figure 5 depicts shear–stress contours for the analyzed beam-to-column joint subassemblages, at the initiation of yield. The state of inelastic shear action indicates

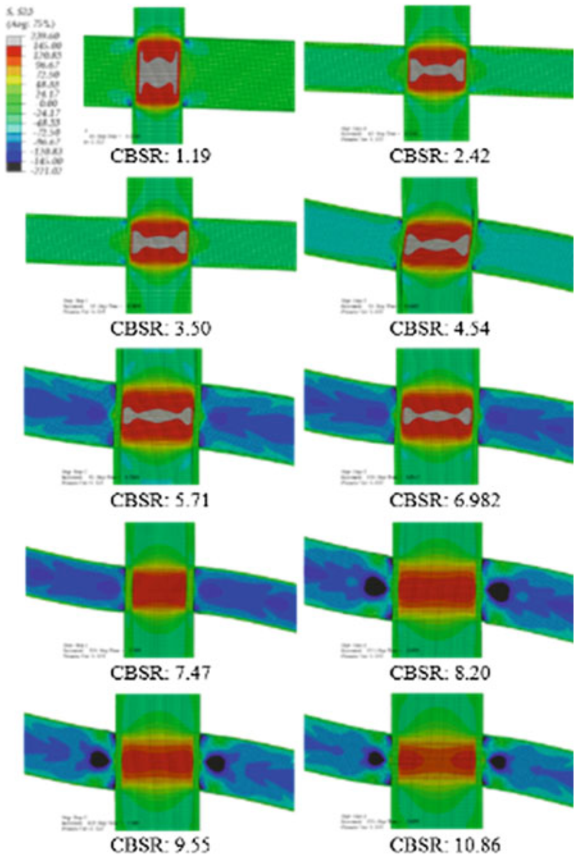


**Fig. 4** Shear–stress contours of beam-to-column joint subassemblages at initiation of yield

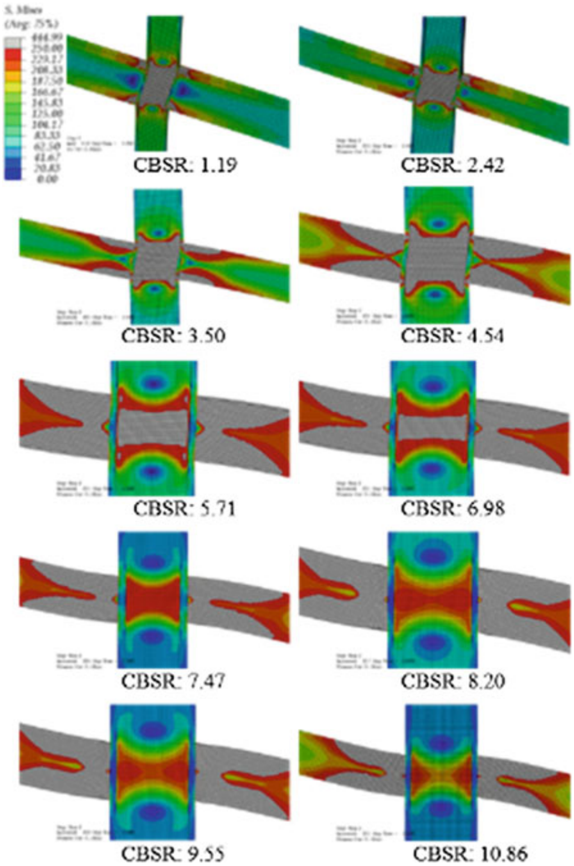


**Fig. 5** von Mises stress contours of beam-to-column joint subassemblages at 4% drift

that, for joints having CBSR lesser than seven, yielding of JPZ initiates at a very small drift level. Inelastic actions in a structure at a drift level of 0.0067 rad are in-acceptable, especially in areas prone to seismic excitations. Table 2 gives the NFEA-based estimates of drifts and beam forces corresponding to those drift levels, at which yielding of three components of a beam-to-column joint occurs. From the table, it can be noted that, up to a CBSR of 4.52, there is no contribution of beams in the inelastic energy dissipation mechanism, and all the inelasticity remains limited to the JPZ region.

Figure 6 shows the force–deformation behaviour of beam-to-column joint subassemblages analyzed for this study. The extent of energy dissipation depends on the sections selected for designing the joint, along with the CBSR.

**Table 2** Yield sequence of components of a beam-to-column moment joint

| S.N. | CBSR | Panel zone yielding | | Beam flange yielding | | Beam plastic hinging | |
|---|---|---|---|---|---|---|---|
| | | %Drift | Force (kN) | % Drift | Force (kN) | % Drift | Force (kN) |
| 1. | 1.19 | **0.644** | **340.97** | 1.099 | 453.62 | – | – |
| 2. | 2.42 | **0.643** | **154.60** | 0.972 | 180.57 | – | – |
| 3. | 3.43 | **0.753** | **139.95** | 1.065 | 177.05 | – | – |
| 4. | 4.52 | 0.840 | 419.93 | **0.699** | **394.39** | – | – |
| 5. | 5.66 | 2.554 | 976.84 | **0.753** | **678.56** | 2.954 | 1014.28 |
| 6. | 7.02 | 3.087 | 773.11 | **0.647** | **484.27** | 2.767 | 751.85 |
| 7. | 7.57 | – | – | **0.768** | **399.93** | 2.568 | 566.81 |
| 8. | 8.43 | – | – | **0.574** | **456.79** | 2.434 | 706.58 |
| 9. | 9.62 | – | – | **0.613** | **401.35** | 2.749 | 623.73 |
| 10. | 10.99 | – | – | **0.614** | **418.92** | 2.954 | 660.30 |

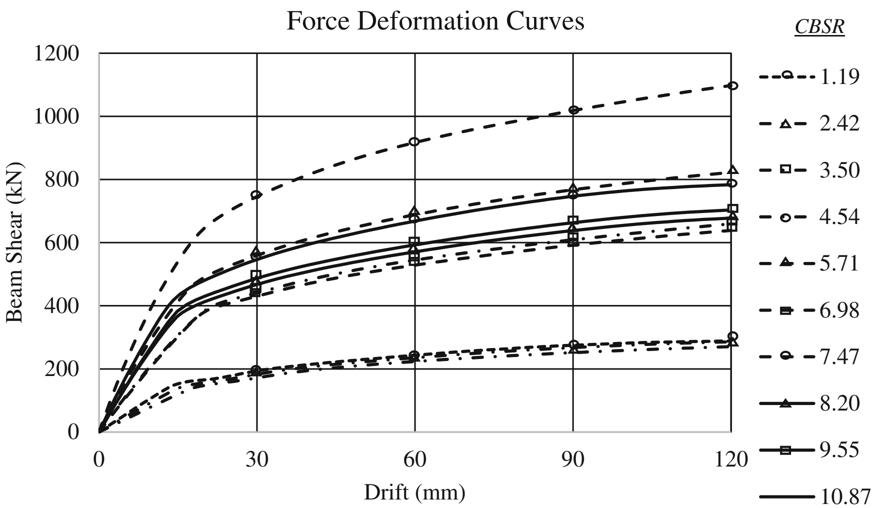


**Fig. 6** Force deformation behaviour of beam-to-column joint subassemblages up to 4% drift

## 4 Conclusions

The seismic behaviour of steel MRFs depends primarily on the CBSR of the beam-to-column joints. The concept of capacity design recommends that the columns of a frame remain undamaged in a moderate level shaking. As joint panel zones are integral part of the columns, inelastic activities in JPZ region shall not be allowed for a moderate level shaking. Following conclusions can be drawn from the study carried out in this paper:

1. The JPZ region in a simple unreinforced beam-to-column moment joint is susceptible to inelastic actions at a drift level of 0.64%. This yield drift is much less than that expected during a moderate level of shaking.
2. The beam end regions of a beam-to-column joint subassemblage shall be allowed form plastic hinges, before shear yielding of JPZ region initiates.
3. In a beam-to-column joint subassemblage, a strong JPZ can be obtained by use of column web stiffeners of sufficient thickness. This thickness of column web region (JPZ) needs to be arrived at, on the basis of strength requirements. Further research is required to determine the column web stiffening strategy.
4. The minimum value of CBSR to prevent inelastic shear yielding of JPZ before the formation of plastic hinges in the beams is about 8.0.

It is recommended that the suitability of steel MRFs in areas of high seismicity is re-evaluated in the light of present conclusions.

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