The strong column-weak beam design philosophy in reinforced concrete frame structures: A literature review

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Abstract: This paper presents a literature review of existing research on the strong column-weak beam design philosophy, which has been widely adopted in the seismic design of frame structures. A comprehensive review of this design philosophy, including its concept and research history, especially factors affecting the accurate calculation of the flexural capacity of beams $\sum M_B$ and the determination of the column-to-beam flexural strength ratio η_C, is presented first. The development of design provisions of four representative countries for this design philosophy is also reviewed. The implementation of strong column-weak beam hierarchy in real structural design is then discussed, and existing problems are pointed out. Finally, different techniques for the seismic retrofit of existing RC frames are reviewed.

Keywords: Strong column-weak beam; structural design; RC frame; seismic retrofit; design code

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1. INTRODUCTION

For the failure mechanism of reinforced concrete (RC) frame structures, the beam-sway mechanism (i.e., with plastic hinges forming first at beam ends) is preferred to the storey-sway mechanism (i.e., with plastic hinges forming first at column ends), as the former generally leads to better seismic performance (Dooley and Bracci 2001; Sunitha et al. 2014; Bai and Ou 2015). Compared with the storey-sway mechanism, more plastic hinges can form in structures which realize the beam-sway mechanism. Therefore, the beam-sway mechanism can provide higher energy dissipation capacity with less demand of ductility on structural components, leading to a more uniform distribution of the storey drift and higher resistance to seismic loads at the structural level (Dooley and Bracci 2001; Sunitha et al. 2014; Bai and Ou 2015). Beam failures are usually localized and will only influence limited parts of the structure. However, column failures may cause progressive collapse of the entire structure and thus can lead to serious consequences. Investigations on the failed structures in great earthquakes have revealed that the collapse of RC frame structures was mainly caused by the failure of the columns (Li 1994; Ye et al. 2008; Chen et al. 2016). To realize the beam-sway mechanism in an RC frame structure when subjected to seismic loading, the enforcement of a strong column-weak beam hierarchy based on the capacity design philosophy (Paulay 1979) has been widely accepted as an effective way. Surana et al. (2018) compared the seismic performances of RC frame structures with/without implementing the strong column-weak beam hierarchy, and the numerical results showed that RC frames without implementing the strong column-weak beam hierarchy exhibited an undesirable storey-sway mechanism while RC frames implementing the strong column-weak beam hierarchy exhibited beam-sway mechanism. A large number of studies have been conducted in order to address the following question: how to introduce the strong column-weak beam design philosophy into structural seismic design reasonably? This paper provides a review of relevant studies to shed light on the invention and development of the strong column-weak beam design philosophy as well as the adoption of this design philosophy in practical design. Firstly, a comprehensive review of the strong column-weak beam design philosophy, including the research history of the strong column-weak beam philosophy and the development of design provisions for this design philosophy, is presented. Then the implementation of strong column-weak beam hierarchy in real structural design is discussed. Finally, different techniques for the seismic retrofit of existing RC frames are reviewed.

2. THE STRONG COLUMN-WEAK BEAM DESIGN PHILOSOPHY

The basic concept of capacity design was raised in 1961 by Blume, Newmark, and Corning (Blume et al. 1961), and adopted in the 1968 version of the Structural Engineers Association of California (SEAOC) recommendations (SEAOC 1968). In 1969, John Hollings, a structural designer in New Zealand, employed the ideas of Blume et al. (1961) and SEAOC (1968) in a design procedure proposed for controlling the inelastic response of concrete buildings (Hollings 1969). The term "capacity design" was used to name this design procedure by a selected team of structural designers of New Zealand's public buildings (Fardis 2018). Tom Paulay publicized and further developed the capacity design into an integral approach for the control of inelastic response, and thus is often called the "father" of capacity design (Fardis 2018). The capacity design method is a type of active seismic design methods, which purposefully guides the failure mechanism of the structure and avoids undesirable failure modes.

The three main aspects covered by the capacity design method are: (1) strong column-weak beam design philosophy: to ensure that the beam-sway mechanism could be realized in frame structures/frame-shear wall structures under seismic loading, that is, plastic hinges could be first formed at beam ends instead of column ends; (2) strong shear capacity-weak flexural capacity design philosophy: to ensure that the shear capacities of structural members (e.g. beams, columns, walls) are larger than their flexural capacities, in order to avoid the occurrence of brittle shear failure of the members; and (3) strong joint-weak member design philosophy: to ensure that the joints are stronger than the structural members, in order to avoid failure in the joints. The present paper is focused on the strong column-weak beam design philosophy.

To achieve the strong column-weak beam hierarchy in an RC frame, the sum of the flexural capacities of the columns at a joint ($\sum M_C$) is required to be larger than that of the beams framing into the joint ($\sum M_B$), as expressed in the following equation:

$$\sum M_C = \eta_C \sum M_B \tag{1}$$

where η_C is called the column-to-beam flexural strength ratio and should be larger than 1. It has been reported that the strong column-weak beam mechanism may be violated due to the following two reasons: (1) underestimation of the flexural capacity of beams (i.e., actual flexural capacity of beams exceeds their flexural capacity calculated in design); and (2) the stipulated value of η_C is too low. Therefore, the related studies on the strong column-weak beam design philosophy have been mainly focused on the calculation of $\sum M_B$ and the determination of a proper η_C, which will be introduced in the following sections.

3. CALCULATION OF THE FLEXURAL CAPACITY OF BEAMS $\sum \boldsymbol{M_B}$

Existing studies on the determination of $\sum M_B$ (Ye et al. 2008; Lin et al. 2009) have mentioned that the value of $\sum M_B$ could be mainly influenced by the existence of cast-in-place floor slabs, infill walls, over-reinforced beam ends, and gravity loads.

3.1 Cast-in-place floor slabs

The effect of cast-in-place floor slabs on the flexural capacity of RC beams has attracted the most attention compared with other influencing factors. Cast-in-place floor slabs connect well with a rectangular beam to form a T-section beam with the rectangular beam acting as the web of the T-section beam and the slabs serving as the flange of the T-section beam, which can significantly enhance the bending stiffness and flexural capacity of the beam. A review of existing studies, including both experimental and numerical investigations, on the effect of cast-in-place slabs on the capacity of RC beams is given below.

3.1.1 Experimental studies

Most of the relevant experimental studies were conducted on RC column-beam joints. Tests on RC column-beam joints conducted by Bertero et al. (1984), Suzuki et al. (1984) and Qi (1986) showed that while the joints were designed in strict accordance with the design codes to prevent failure in joints, plastic hinges formed first at the column ends. The reason was an unexpected increase in the flexural capacity of the beams mainly caused by cast-in-place floor slabs.

Leon (1984) reported two contrast specimens (a column-beam joint without a slab and another one with a slab) to clarify the effect of cast-in-place floor slab on the behavior of RC joints. Again, there were no design provisions at that time to account for the increase in the flexural capacity of beams caused by slabs. Test results showed that the presence of a cast-in-place floor slab significantly affected the strength and behavior of the joints: the plastic hinges formed first at the beam ends in the joint which had no slab, while the plastic hinges formed first at columns ends in the joint which had a slab.

Durrani and Zerbe (1987) tested a total of six three-quarter-scale joints under cyclic lateral loading to study the effect of cast-in-place slabs on the behavior of exterior joints. The test results showed that the cast-in-place floor slab had a significant effect on the strength, stiffness and energy dissipation characteristics of the joints. Thus it was strongly suggested that the effect of cast-in-place floor slabs should be considered in the design of joints.

Durrani and Wight (1987) tested three interior joints to study the effect of cast-in-place slabs on the behavior of interior joints. The test results indicated that the cast-in-place slabs obviously affected the behavior of interior joints: at a drift level of 1.5%, the steel reinforcement in the slab began to yield, while all the steel reinforcement within the entire width of the slab had yielded at a drift level of 4%. Therefore they concluded that the contribution of the slab to the flexural capacity of the beam cannot be ignored.

Researchers from Tongji University and the China Academy of Building Research in collaboration with those from Japan, New Zealand and the United States tested six full-scale two-way joints (Tang 1989), and reported that the cast-in-place floor slab significantly enhanced the negative flexural capacity of the beams.

French and Moehle (1991) conducted analysis on the collected test data of 20 beam-slab-column joints (13 interior joints and 7 exterior joints). The results showed that the predicted strength of interior joints with the effect of the cast-in-place floor slab ignored was less than the test result by an average of 25%, while the predicted strength of exterior joints with the effect of the cast-in-place floor slab ignored was less than the test result by an average of 17%.

Jiang et al. (1994) tested two contrast specimens, with one joint having a cast-in-place floor slab and the other one having no slab. The test results showed that due to the contribution from the cast-in-place floor slab, the negative flexural capacity of the beam increased by as much as 30%.

Zhen et al. (2009) tested three groups of RC joints with different reinforcement schemes under cyclic lateral loading. Each group included a joint without a cast-in-place floor slab and one/four/two joints with a cast-in-place floor slab. Test results showed that the strengths of joints with a cast-in-place floor slab in group 1, 2 and 3 were respectively about 1.6, 2.0 and 2.3 times of those of the corresponding joints without a cast-in-place floor slab.

Gunasekaran and Ahmed (2014) tested four half-scale joints under cyclic lateral loading: one without a cast-in-place floor slab and others with slabs of varying slab parameters. Test results indicated that the cast-in-place floor slab can contribute significantly to the flexural resistance of the beam under positive/negative bending.

In addition to the tests on RC column-beam joints, some researchers conducted experimental studies on RC frames. In the early 1980s, experimental studies on a full-scale 7-storey RC frame were jointly conducted by the researchers from the United States and Japan (Durrani and Wight 1982; Otani et al. 1984; JTCC 1988). The studies showed that in the RC frame under lateral loading, cast-in-place floor slabs could lead to a considerable increase in the flexural capacity of the beam. Such an increase was not taken into consideration in the design of the frame as such consideration was not available in design provisions at that time. Therefore, failure of the test frame was controlled by shear failure at joints.

Qi and Pantazopoulou (1991) conducted a test on a quarter-scale single-storey RC frame with cast-in-place floor slabs under cyclic lateral loading. The test results showed that cast-in-place slabs significantly increased the flexural capacity of the beams, especially at the interior support.

Ning et al. (2014) tested two 2/3 scale 2-storey 1x2-span (the two span numbers in the two perpendicular directions in the plane) spatial RC frames, with one having cast-in-place floor slabs and the other one having no slabs. The test results showed that due to the contribution from the cast-in-place floor slabs, the failure pattern of the RC frame without slabs was beam-sway mechanism while that with slabs turned into storey-sway mechanism.

All the experimental studies above (a typical test setup is shown in Fig. 1) showed that the cast-in-place floor slab can significantly enhance the flexural capacity of the beam, which should be taken into consideration in the design of RC frames. However, most of these experimental studies focused on RC beam-column joints with/without cast-in-place floor slab, whose behavior may be different from that of spatial RC frames. The experimental studies on spatial RC frames with/without cast-in-place floor slabs are very limited and more relevant studies need to be conducted in the future.

3.1.2 Numerical studies

In addition to experimental investigations, a large number of numerical studies on the effect of cast-in-place floor slabs on the seismic performance of RC frames have been conducted. Guan and Du (2005) conducted pushover analysis of a piece of 3-storey-3-span RC frame using SAP2000 (1998). After the 2008 Wenchuan earthquake in China, Lin et al. (2009) conducted elastic-plastic time history analysis of a 6-storey RC frame structure damaged in the earthquake using MSC.Marc (2005). Comparison between two 3-dimentional (3-D) models (one is pure frame and the other is frame with cast-in-place floor slabs) was carried out. Gao and Ma (2009) conducted pushover analyses of two 6-storey 4x4-span RC frames (one with floor slabs and one without floor slabs) using SAP2000; Yang (2010) conducted pushover analyses of five 6-storey 4x4-span RC frames with different slab widths using SAP2000; Chen (2010) conducted elastic-plastic time history analyses of two 6-storey 6x3-span RC frames (one with floor slabs and one without floor slabs) using

SAP2000. Guo (2012) designed a 3-storey 3x4-span RC frame, established three FE models and conducted pushover analyses using SAP2000: one with cast-in-place slabs and slab reinforcement, one with cast-in-place slabs but without slab reinforcement and one without slabs. Ning et al. (2016) conducted pushover analyses of two 2-storey 1x2-span RC frames (one with floor slabs and the other one without floor slabs) using ABAQUS, and the numerical results agreed well with the test results of the same RC frames tested by Ning et al. (2014).

Details of the above numerical analyses are given in Table 1. All these numerical results indicated that cast-in place floor slabs could significantly increase the negative flexural capacity of the beams and could result in the weak column-strong beam mechanism in RC frames. However, the accuracies of most of these numerical studies were not verified with test results. It would be better if combined experimental and numerical studies can be conducted, and the numerical models whose accuracies are verified using the test results can be used for further parameter studies to better understand the structural behavior of RC frames with floor slabs.

3.1.3 Determination of effective flange width

A large number of experimental studies (e.g. Jiang 1994; Bijan and Aalami 2001; Huang et al. 2001) have indicated that the stresses of steel bars in a cast-in-place floor slab in tension/compression are not evenly distributed along the width direction of the beam. Instead, the stress in a steel bar in the floor slab deceases with an increase in the distance between the steel bar and the beam, due to the well-known shear lag effect. Therefore, only steel bars within a limited range of width away from the beam can reach their yield strength at the failure of the beam (Wu et al. 2002; Wang et al. 2009; Zhen et al. 2009). In order to quantify the effect of a cast-in-place floor slab on the flexural capacity of the beam, an effective flange width (b_f) has been proposed by previous researchers in the calculation of contribution from a cast-in-place floor slab to the flexural capacity of the beam supporting it (Wu et al. 2002; Wang et al. 2009). It is assumed that all longitudinal steel bars in the cast-in-place floor slab within the effective flange width can be equally strained in the bending of the beam. The suggested values of the effective flange width of floor slabs for interior and exterior joints (Durrani and Zerbe 1987; French and Moehle 1991; Li 1994; Jiang et al. 1994; Wu et al. 2002; Wang et al. 2009; Zhen et al. 2009; Yang 2010; Sun 2010; Qi et al. 2010; He 2010; Ning et al. 2016) are summarized in Table 2.

It can be seen from Table 2 that the factors which can influence the effective flange width include the inter-story drift angle (γ) (i.e. inter-story drift divided by the story height), joint types (i.e. interior joints and exterior joints), slab thickness (t), beam height (h), effective span of beam (l_0) and clear distance between two adjacent beams. A large number of the existing studies (French and Moehle 1991; Zhen et al. 2009; Sun 2010; Qi et al. 2010; He 2010; Ning et al. 2016) examined the effective flange width when inter-story drift angle is equal to 1/50. Most existing studies only paid attention to interior joints, while four studies (Zhen et al. 2009; Sun 2010; Qi et al. 2010; Ning et al. 2016) proposed effective flange widths for both interior joints and exterior joints. It was found that the effective flange width for interior joints is usually larger than that for exterior joints if the other parameters are the same. In addition, most formulas proposed to calculate the value of effective flange width use the slab thickness as the main parameter (e.g. Li 1994; Jiang et al. 1994; Wu et al. 2002; Wang et al. 2009; Yang 2010; He 2010), while several formulas related the effective flange width to more factors such as the beam height, the effective span of beam and the clear distance between two adjacent beams and so on (e.g. French and Moehle 1991; Zhen et al. 2009; Sun 2010; Qi et al. 2010), which are more comprehensive. As can be seen from Table 2, the suggested effective flange widths by researchers for either interior joints or exterior joints are not small, which also implies that the enhancement of cast-in-place floor slab for the flexural capacity of the beam could be significant. For a typical case (b_c=300 mm; b=250 mm; h=500 mm; l_0=6 m; t=100 mm; s=3 m), the suggested effective flange widths by the researchers are calculated and listed in Table 2. It can be seen from Table 2 that the suggested effective flange widths vary largely among different studies, and a widely accepted effective flange width has not been established.

3.2 Infill walls

Ye et al. (2008) studied RC frames damaged in the 2008 Wenchuan earthquake in China and analyzed the factors that caused the violence of the strong column-weak beam mechanism. The effect of infill walls, which was not fully considered in structural design, was found to be one of the main causes. In most real structures, infill walls usually stand directly on the beams, which would cause the following effects (Ye et al. 2008): (1) infill walls can increase the stiffness and flexural capacity of the beam and reduce the deformation of the beam; (2) infill walls will be involved in the seismic performance of the overall structure, increasing the stiffness of the storeys with infill walls, leading to a non-uniform stiffness distribution of the structure (i.e. the stiffness of the storeys with infill walls is larger than that of the storeys without infill walls), rendering storeys with no infill walls weak layers (usually at the ground floor) and thus resulting in the formation of the storey-sway mechanism at the ground floor; infill walls would also lead to an irregular distribution of the structural plane stiffness and cause a torsional effect; (3) due to the existence of infill walls, the total stiffness of the structure would increase, leading to a decrease in the basic period of the structure by about 40%-60% and thus an increase in the seismic loading; and (4) infill walls would affect the internal force distribution and failure mode of RC frames. For example, the lateral deformation of a column can be restricted by the infill walls and thus the column would become a short column (Xiong 2011; Li 2015). Ye et al. (2008) concluded that the effect of infill walls on the seismic performance of the whole structure was very complicated and should be considered in structural design.

Many studies were conducted to quantitatively investigate the effect of infill walls on the behavior of RC frames. The details of these studies are listed in Table 3.

Lin et al. (2009) conducted elastic-plastic time history analysis on a 6-storey 3x9-span RC frame structure damaged in the earthquake area by using MSC.Marc (2005). Comparisons between three schemes (pure frame, frame with cast-in-place floor slabs, and frame with both cast-in-place floor slabs and infill walls) were carried out. The analysis results indicated that infill walls may significantly change the failure mechanism of the RC frame and the storey-sway mechanism may easily occur for a structure with non-uniformly distributed infill walls. Lin et al. (2009) suggested that the effects of infill walls should be considered in seismic design of RC frames and structural elastic-plastic numerical analyses of RC frames should also take into account infill walls.

Chen (2010) designed four 6-storey 6x3-span RC frames and established FE models for them: one pure frame, one frame with floor slabs, one frame with both cast-in-place floor slabs and infill walls, and one frame with both cast-in-place floor slabs and infill walls except the ground floor. Results of linear and non-linear time history analyses conducted on these four frame models indicated that the existence of infill walls affected the failure mode of the structure. In particular, the non-uniform layout of infill walls led to the formation of a weak layer in the frame and the storey-sway mechanism.

Xiong (2011) established FE models for two 5-storey 2-span RC frames (one pure frame and one frame with infill walls) and conducted pushover analyses of these two frames. Analysis results indicated that the failure of RC frames without infill walls for the ground level would occur at the ground level.

Shi (2012) tested two 1/4-scale 3-storey 2x2-span RC frames, one pure frame and one frame with infill walls, and then conducted elastic-plastic time history FE analyses of these two frames using PERFORM-3D (2011). Both test and numerical results showed that infill walls could significantly change the internal force distribution of the structure and increase bending moments at the column ends, leading to the failure of columns prior to the failure of beams.

Yuen and Kuang (2015) established FE models for six 2-storey 2-span RC frames with different infill configurations (one pure frame, one frame with full infill walls, one frame with 2/3-storey-height infill walls, one frame with infill walls except the ground floor, one frame with full infill walls and window openings, and one frame with full infill walls and door openings) and conducted elastic-plastic time history FE analyses of these six frames. The numerical results showed that columns of RC frames with infill walls suffer much greater damage than the adjacent beams, and therefore the strong column-weak beam hierarchy may not be realized in RC frames with infill walls.

Fiore et al. (2016) conducted pushover analyses of three 4-storey 5x3-span RC frame models: one pure frame, one frame with infill walls, and one frame with infill walls except the ground floor. The analysis results indicated that infill walls may change the failure mechanism of the RC frame.

Mohammed and Güneyisi (2018) established FE models for 2-, 4-, 6- and 8-storey RC frames without infill walls or with four different infill wall arrangements, and conducted pushover analyses of these frames. Analysis results indicated that the infill walls caused change of the distribution of plastic hinges and failure mechanism of the structure.

Several studies have also studied the effect of infill walls on the progressive collapse performance of RC frames (Li et al. 2016; Shan et al. 2016; Hadi et al. 2018). It can be concluded from their studies that the existence of infill walls can enhance the progressive collapse capacity of RC frame, but may reduce the ductility and change the failure mode of the RC frame, which indicates that the effect of infill walls should be considered in progressive collapse design of RC frames.

It can be seen from Table 3 that the modeling methods of infill walls in RC frames used by different researchers are quite different and a widely accepted modeling method of infill walls has not been established, which can be a future study topic.

3.3 Over-reinforced beam ends

In structural design, over-reinforcement of beam ends is quite usual and could be caused by the following reasons (Ye et al. 2008; Liu 2004; Wei et al. 2007): (1) the reinforcement of beam ends may be designed based on the bending moment at joint centre rather than at the beam end (i.e., omission of the width of column); (2) the reinforcement of the beam may be controlled by the limit of deformation or crack width rather than strength; (3) when the moments at the two beam ends of a joint is not equal, for ease of construction, the reinforcement at both beam ends is usually designed based on the larger value of the two moments; and (4) the real cross-sectional area of reinforcement is usually enlarged to certain extent by the designer to achieve a "safer" design. The adverse effects of the above factors can be avoided if the calculation of ΣM_B in Eq. 1 is based on the actual reinforcement. However, the calculation of ΣM_B is usually based on the design bending moments at the beam ends rather than the actual reinforcement (e.g. GB-50011 2010 or older versions; ACI 318 1983). A summary of existing studies on the effect of over-reinforced beam ends in RC frames is given in Table 4. By using PL-AFJD (Yang 2000), Lei (2002) conducted elastic-plastic time history analyses of three RC frames, for two of the frames, the flexural capacities of beams and columns were calculated based on the actual reinforcement, and for one of the frames, the flexural capacities of beams and columns were calculated based on the design moments; Liu et al. (2004) conducted elastic-plastic time history analyses of two RC frames, for one of the frames, the flexural capacities of beams and columns were calculated based on the actual reinforcement, and for the other frame, the flexural capacities of beams and columns were calculated based on the design moments. By using OpenSees (2009), Han et al. (2010) conducted push-over analyses on two RC frames, with one having no over-reinforced beam ends and the other one having over-reinforced beam ends (the beam reinforcement at beam ends was increased by 10%). Analysis results showed that the RC frames which did not have over-reinforced beam ends exhibited beam-sway mechanism, while storey-sway mechanism was formed in RC frames which had over-reinforced beam ends.

3.4 Gravity loads

The conventional methodology for quasi-static cyclic tests on RC beams/beam-column joints does not consider the gravity load effects. Actually, due to the existence of gravity loads, two kinds of plastic hinges may form in RC beams subjected to seismic loading: reversing and unidirectional plastic hinges (Fenwick et al. 1999). When the ratio between the bending moments induced by gravity load and that induced by the seismic action is low, plastic hinges will form at the beam ends, which are reversing plastic hinges. However, when the ratio between the bending moments induced by gravity load and that induced by the seismic action is high, negative moment plastic hinges will form at the beam ends while positive moment plastic hinges will form in the beam spans, which are unidirectional plastic hinges (Fenwick et al. 1999). Compared with the reversing plastic hinges whose behavior is predictable, the behavior of unidirectional hinges is unpredictable and quite complex, and RC beams with unidirectional hinges will exhibit higher damage levels (Dhakal and Fenwick 2008; Gião et al. 2019). Therefore, a suitable seismic design strategy is necessary to prevent the formation of unidirectional plastic hinges, which is not considered in the design codes except the New Zealand Code (NZC-1170 2004). Some studies have been conducted to investigate the effect of gravity loads on the seismic performance of RC frames (Megget and Fenwick 1989; Dhakal and Fenwick 2008; Gião et al. 2014; Gião et al. 2019; Muhaj et al. 2019), and it is suggested from these studies that the failure mechanism associated with the formation of plastic hinges in the beam spans should be avoided through the implementation of detailing design strategies, in order to more successfully achieve the strong column-weak beam hierarchy.

4. DETERMINATION OF COLUMN-TO-BEAM FLEXURAL STRENGTH RATIO (η_C)

A proper value of the column-to-beam flexural strength ratio η_C in Eq. 1 is very important to achieve the strong column-weak beam mechanism in RC frames. By now, there have been a large number of studies on the determination of η_C, which are summarized in Table 5 and explained below.

Only one of the studies was experimentally based. Xu et al. (1986) conducted an experimental study on a piece of 3-storey 2-span RC pure frame (i.e., without slabs) under cyclic lateral loading to investigate the relationship between the strengths of columns and beams framing into a joint. The test results showed that η_C of each joint in the frame was between 1.42 and 2.86 and the frame achieved beam-sway mechanism with good ductility.

Most of the relevant studies adopted the numerical methods. The common method adopted by the researchers was determining the required η_C which could make the frames achieve the strong column-weak beam hierarchy through an elastic-plastic time history analyses or pushover analyses on the established FE models of RC frames.

The elastic-plastic time history analyses of RC frame models conducted by Xu et al. (1986) indicated that an η_C of 1.25, which was the value recommended by the Chinese design manual (Chinese Academy of Building Research 1981), was not sufficient for a frame to achieve the strong column-weak beam hierarchy.

Wei et al. (2003) designed 6 RC pure frames of Seismic Grade (SG) 2 in a Seismic Precautionary Intensity (SPI) 8 region [i.e. the height of frames in this region is not larger than 30 m following GB-50011 (2001)] in China and carried out elastic-plastic time history analyses of these frame models using the nonlinear dynamic analysis program PL-AFJD (Yang 2000). The results indicated that for RC frames of SG 2 in an SPI 8 region in China, an η_C value of 1.2, which is prescribed in the old design code GB-50011 (2001), was not sufficient, and an η_C value of 1.4-1.5 was suggested.

Wei et al. (2007) designed five pieces of 6-storey 3-span RC frames in different SPI regions in China following the Chinese design code GB-50011 (2001) and carried out elastic-plastic time history analyses of these frames using FW-EPA (Wei 2005). The results showed that the frame of SG 1 in an SPI 9 region [i.e. the height of frame in this region is not larger than 25 m following GB-50011 (2001)] achieved the beam-sway mechanism, while the storey-sway mechanism happened in the frames of SG 2 in an SPI 8 region and SG 3 in an SPI 7 region [i.e. the height of RC frame in this region is not larger than 30 m following GB-50011 (2001)]. η_C=1.3 was suggested by Wei et al. (2007) for frames of SG 3 in an SPI 7 region. For frames of SG 2 in an SPI 8 region, Wei et al. (2007) suggested η_C=1.0, with the calculation of $\sum M_B$ being based on the actual reinforcement.

Tao (2010) established an FE model of 2-storey 3x3-span RC frame with cast-in-place floor slabs using ANSYS (2007), and increased the value of η_C gradually until the beam-sway mechanism was achieved. Based on the analysis results, a value of η_C=1.7 was recommended.

Yang (2010) established 6 RC frame models with cast-in-place floor slabs and different η_C values and carried out static nonlinear analyses and nonlinear time-history analyses on these frames using SAP2000 (1998) to determine the reasonable value of η_C. Only RC frames of SG 2 in an SPI 8 region in China were taken into consideration in the analyses. The analysis results showed that the requirement η_C=1.2 in Chinese code GB-50011 (2001) was not sufficient for frames to achieve the beam-sway mechanism, and a value of η_C ranging from 1.6 to 2.0 was suggested by Yang (2010).

Ye et al. (2010) carried out elastic-plastic time history analysis on RC pure frames excited by 20 strong ground motions using THUFIBER (Lu et al. 2006) to study the required η_C values for the frames to achieve the beam-sway mechanism. Analysis results showed that the required η_C value increased with the earthquake intensity. Based on the analysis results, the values of η_C should be 2.0, 1.7 and 1.4 for RC frames respectively of SG 1, 2 and 3 in China. However, considering that the values of η_C stipulated in the latest Chinese seismic code (GB-50011 2008) for RC frames of SG 1, 2 and 3 are respectively 1.4, 1.2 and 1.1, moderate values of 1.7, 1.5 and 1.3 were suggested by Ye et al. (2010) for RC frames of SG 1, 2 and 3 respectively.

Sun (2010) conducted dynamic time history analysis on a series of RC frames with cast-in-place floor slabs and different η_C values using ABAQUS (2006) to study their displacements, storey drifts and distributions of plastic hinges under a rare earthquake. A value of η_C ranging from 1.8 to 2.0 was suggested by Sun (2010) for RC frames of SG 2 in China (GB-50011 2001).

Yang (2011) established a group of 6-storey RC frame models with cast-in-place floor slabs and different η_C values, and conducted elastic-plastic time history analysis on these frames under three-dimensional earthquake actions using MSC.Marc (2005). A value of 2.4, 2.1, 1.9 and 1.6 were suggested for η_C of RC frames of SG 1, 2, 3 and 4 respectively in China (GB-50011 2010).

Yang (2012) established FE models of three 5-storey RC frames (with cast-in-place floor slabs) of SG 3 in China respectively with three different values of η_C: 1.3 [according to Chinese code GB-50011 (2010)], 1.4 and 1.5. Results from the pushover analyses carried out on these frames showed that the strong column-weak beam hierarchy was achieved when η_C=1.5.

Sunitha et al. (2014) established two 5-storey and one 10-storey RC pure frame models with various values of η_C and conducted nonlinear static pushover analysis on these three frames using SAP2000 (1998) to demonstrate the effect of η_C on the seismic behavior of frames. Analysis results showed that the η_C value required to achieve the beam-sway mechanism in these RC frames was between 2.5 and 3.0.

By using SAP2000 (1998), Sargar and Bhusari (2018) conducted pushover analyses on three 3-storey RC pure frame models (frame A having η_C less than 1.4 and frames B and C having η_C more than 1.4). Analysis results showed that $\eta_C>1.4$ should be maintained to achieve the beam-sway mechanism in these RC frames.

In addition to the experimental studies and numerical studies, some researchers conducted theoretical studies to determine the proper value of η_C.

Dooley and Bracci (2001) evaluated the seismic performance of a 3-storey frame and a 6-storey frame with various η_C values (0.8, 1.0, 1.2, 1.6, 2.0, 2.4) using probabilistic measures. The results showed that an η_C value of 1.2, which was the requirement of ACI 318 (1999), led to only a 10% probability of preventing the formation of storey-sway mechanism, while an η_C value of 2.0 led to a much higher probability (roughly 80%) of preventing the formation of storey-sway mechanism. So η_C=2.0 was suggested by the authors.

Ma and Chen (2005) analyzed the reliability of strong column-weak beam design in a 6-storey RC pure frame with different values of η_C ranging from 1.0 to 2.0 and recommended a value of 1.6.

Cai et al. (2007) analyzed the failure probability of strong column-weak beam design for single RC joints using the theory of reliability, and conducted Monte Carlo simulation on a 3-storey and a 6-storey RC frames with floor slabs. Analysis results indicated that the acceptable probability of achieving the strong column-weak beam mechanism can be obtained if η_C is no less than 2.0.

Based on structural reliability theory, Xia (2009) studied the strong column-weak beam design of RC frames following the Chinese design code GB-50011 (2001). According to the analysis results, Xia (2009) gave some advice on the strong column-weak beam design method, with η_C=1.4, 1.3 and 1.2 being suggested respectively for RC frames of SG 2 in an SPI 8 region, SG 2 in an SPI 7 region [i.e. frames in this region whose height is larger than 30 m following GB-50011 (2001)], and SG 3 in an SPI 7 region.

It can be concluded from Table 5 that, with different analysis method adopted (e.g. numerical analysis and theoretical analysis), the suggested values of η_C by the researchers from different countries (e.g. China, United States and India) are mostly much larger than 1.0. Moreover, it can be seen from Table 5 that there is a large variation among the suggested values of η_C by different researchers. This may be mainly due to the suggested values of η_C by different researchers are case-dependent, which indicates that more systemic studies (i.e. considering the effects of number and heights of storeys, number and lengths of spans, vertical loads etc.) need to be conducted to determine more reasonable and widely acceptable value of η_C. It should be noted that the η_C suggested based on the experimental and theoretical studies may be more reliable than that based on the numerical studies.

5. DEVELOPMENT OF DESIGN PROVISIONS FOR THE STRONG COLUMN-WEAK BEAM DESIGN PHILOSOPHY

In this section, design provisions to implement the strong column-weak beam design philosophy in the design codes from New Zealand [NZS-3101 (2006) and previous versions], the United States [ACI 318 (2019) and previous versions], Europe [Eurocode 8 (2004) and previous versions] and China [GB-50011 (2010) and previous versions] are reviewed.

5.1 New Zealand

As mentioned in Section 2, the term “capacity design” was originally proposed in New Zealand. Till now, New Zealand’s structural design code is one of the most advanced codes in the world.

NZS-95 (1935) for the first time gave the seismic design method in New Zealand after a number of major earthquakes in late 1920s and early 1930s, while NZS-4203 (1976) for the first time adopted the capacity design method. NZS-3101 (1982) adopted the capacity design method and provided many requirements for capacity design (Gregory et al. 2011; Fenwick and MacRae 2009).

The consideration of the contribution from the cast-in-place floor slab within the effective flange width to the stiffness and flexural capacity of the beam first appeared in NZS-3101 (1982), and was improved in NZS-3101 (2006) to cover more factors. The stipulations on the strong column-weak beam design philosophy for RC frames in NZS-3101 are given in Table 6.

5.2 United States

The building code of the United States is one of the world’s widely referenced codes

(Liu 2006). This section is focused on the American Concrete Institute (ACI) Code [ACI 318 (2019) and previous versions], which is the most widely used building code in the United States.

ACI 318 (1971) stipulated that the sum of the flexural capacities of the columns at a joint should be larger than that of the beams at the joint (i.e., the value of η_C in Eq. 1 should be larger than 1.0), while ACI 318 (1983) increased the value of η_C to be 1.2. ACI 318 (2002) and later versions (ACI 318 2005, 2019) stipulated that the contribution of the cast-in-place floor slab within the effective flange width to the stiffness and flexural capacity of the beam should be taken into account. A comparison between different versions of ACI 318 in terms of the strong column-weak beam design philosophy is given in Table 7.

5.3 Europe

In both of the two versions of Eurocodes (Eurocode 8 1995, 2004), the strong column-weak beam design philosophy is adopted and the contribution of cast-in-place floor slabs within the effective flange width to the stiffness and the flexural capacity of beams should be taken into account. The stipulations on the strong column-weak beam design philosophy for RC frames in Eurocode 8 (1995, 2004) are listed in Table 8. The column-to-beam flexural strength ratio η_C is stipulated to be 1.3. It should be noted that the calculation of $\sum M_B$ is based on the design reinforcement rather than the actual reinforcement of the beam.

5.4 China

The strong column-weak beam design philosophy was adopted in GBJ11-89 (1989) for the first time and developed in the subsequent versions. Comparisons of stipulation on the strong column-weak beam philosophy of RC frames in Chinese design codes are given in Table 9. In GBJ11-89 (1989) and GB-50011 (2001), the contribution of cast-in-place slab to flexural capacity of the beam was not considered. After the 2008 Wenchuan Earthquake, the value of η_C was significantly increased in the latest version of Chinese seismic design code GB-50011 (2010), and RC frame structures of SG 1 need to meet the following requirement:

$$\sum M_C = 1.2 \sum M_{Bua} \quad (2)$$

where $\sum M_{Bua}$ is based on the actual reinforcement and the characteristic strength of materials, with the contribution of cast-in-place floor slabs to both the stiffness and the flexural capacity of beams considered for the first time in Chinese design codes.

6. IMPLEMENTATION OF THE STRONG COLUMN-WEAK BEAM DESIGN PHILOSOPHY IN STRUCTURAL DESIGN

6.1 Existing problems

Structures designed using design codes which do not adopt the capacity design method cannot or can hardly achieve the strong column-weak beam hierarchy. Storey-sway mechanism rather than beam-sway mechanism will form in such structures. Despite that the newer design codes adopt the capacity design method and specify the value of η_C, studies of failed structures after major earthquakes have shown that the beam-sway mechanism rarely occurred (ATC-40 1996) because most of the failed frames were designed according to codes (generally previous codes) which do not or do not adequately enforce the strong column-weak beam requirement. Moreover, although the current design codes investigated in Section 5 well consider the effect of cast-in-place floor slabs on the flexural capacity of the beam, the value of η_C given in these design codes are still smaller than those suggested by researchers (as discussed in Section 4). Therefore, structures designed based on the latest versions of design codes probably still cannot completely achieve the strong column-weak beam hierarchy.

For example, statistical results based on 48 frame structures which suffered damage from the 1976 Tangshan earthquake in China showed that most frame structures with cast-in-place floor slabs failed in the storey-sway mechanism, while frame structures without cast-in-place floor slabs failed in the beam-sway mechanism (Li 1994). In the magnitude (Ms) 8.0 Wenchuan earthquake in China in 2008 (Chinese Academy of Building Research 2008), failure of cast-in-place RC frames commonly occurred at column ends (Fig. 2); the beam-sway mechanism was normally found only in frames with no floor slabs or with precast floor slabs (Fig. 3). The prevalence of column end failures has been attributed to major deficiencies in GB-50011 (2001) and its previous versions: the versions before GB-50011 (2001) do not include the capacity design, while GB-50011 (2001) does; GB-50011(2001), however, does not consider the contribution of the cast-in-place slab in tension to the flexural capacity of the beam in negative bending in its specification (Lin et al. 2009; Zhou et al. 2013). As a result, it can be expected that in many RC frames in the Chinese mainland, the beams are stronger than the columns at a joint. The new Chinese seismic design code (GB-50011 2010), which came into force in December 2010, requires the consideration of the contribution of the cast-in-place slab to the beam flexural capacity in addition to the adoption of higher flexural strength ratios. Many existing RC buildings cannot meet these new requirements. Moreover, the suggested values of η_C by some researchers summarized in Table 6 are still much larger than those stipulated in GB-50011 (2010), and only for RC frame structures of SG 1, the calculation of $\sum M_B$ needs to be based on the actual reinforcement which considers the contribution of reinforcement in the slab; for RC frames of other SGs, the calculation of $\sum M_B$ is still based on the designed reinforcement which does not consider the contribution of reinforcement in the slab. This situation indicates that structures built after 2010 probably still cannot completely meet the requirement of achieving the strong column-weak beam mechanism, which has been verified by some studies (Duan and Hueste 2012; Lin et al. 2015; Chen et al. 2016). Duan and Hueste (2012) conducted both push-over analysis and dynamic time-history analysis on a typical 5-storey RC frame model to evaluated the seismic performance of RC frames designed according to GB-50011 (2010), and the analysis results indicated that the RC frame has the potential to exhibit a soft first story failure mechanism. After the Ms 6.5 Ludian earthquake in China in 2014, Lin et al. (2015) and Chen et al. (2016) conducted field investigations to evaluate the damage to buildings. A common failure mode was observed that most of the RC frames were badly destroyed due to the collapse of the first storey, which indicated that the RC frames designed based on GB-50011 (2010) still failed to achieve the strong column-weak beam hierarchy.

In other countries or regions, similar threats due to the violation of the strong column-weak beam hierarchy in the older existing buildings may also exist. For example, buildings designed in accordance with ACI 318 (1983) or older versions which did not consider the contribution of the cast-in-place slab in tension to the flexural capacity of the beam in negative bending are likely to violate this hierarchy.

6.2. Seismic retrofit of existing RC frames

To achieve the strong column-weak beam hierarchy in existing cast-in-place RC frames where such a hierarchy has not been satisfied, strengthening of columns might be an option. Common column retrofitting methods include concrete jacketing (e.g. Thermou et al. 2007; Vandoros and Dritsos 2008; Campione et al. 2014; Deng et al. 2019), steel jacketing (e.g. Xiao and Wu 2003; Nam et al. 2009; Choi et al. 2010; Wang and Su 2012; He et al. 2017) and FRP jacketing (e.g. Teng et al. 2002; Xiao 2004; Teng et al. 2016a; Pan et al. 2018). The former two methods may lead to increases in mass and/or stiffness and then increases in seismic forces, while FRP jacketing has been widely used in recent years as a simple but effective method for column strengthening. However, the strength enhancement due to FRP jacketing may be small, especially when FRP jacketing is applied to confine non-circular columns (Lam and Teng 2009; Teng et al. 2016b). And even when column strengthening can be sufficient, the location of failure may simply shift from column ends to the foundation and/or beam-column joints. Therefore, column strengthening alone is often not sufficient enough to change the strength hierarchy; joint strengthening is always needed. The strengthening techniques for RC beam-column joints include (Engindeniz et al. 2005): epoxy repair (e.g. French et al. 1990; Filiatrault and Lebrun 1996; Karayannis et al. 1998), removal and replacement (e.g. Lee et al. 1977; Tsonos 2001), concrete jackets (e.g. Hakuto et al. 2000; Tsonos 2001; Tsonos 2003), reinforced masonry blocks (e.g. Aycardi et al. 1994; Bracci et al. 1995a, b), steel jackets and external steel elements (e.g. Hoffschild et al. 1995; Biddah et al. 1997; Ghobarah et al. 1997), FRP jackets (e.g. El-Amoury and Ghobarah 2002; Ghobarah and Said 2002; Antonopoulos and Triantafillou 2003), and so on. Among all these joint strengthening techniques, strengthening using FRP jackets attracts the most attentions nowadays, as externally bonded FRP composites can avoid some important limitations of other strengthening techniques such as increases in member sizes and difficulties in construction (Bousselham 2010; Sezen 2012; Seifi et al. 2017; Mostofinejad and Hajrasouliha 2019).

Instead of column strengthening, a novel seismic retrofit method for cast-in-place RC frames which violate the strong column-weak beam hierarchy has been proposed by Teng et al. (2013). This method is based on the concept of Beam-end Weakening in combination with FRP Strengthening (referred to as the BWFS method hereafter for simplicity), to implement the strong column-weak beam hierarchy. The technique is based on the weakening of the flexural capacities of the T-section beams at a joint, particularly when the flange (i.e. the cast-in-place slab) is in tension. The general concept of local weakening is not new in seismic retrofit or design. In steel structures, a typical weakening technique for new structures and seismic retrofit is adopting the dog-bone design to ensure a weak beam-strong connection strength hierarchy (Popov et al. 1998; Wang et al. 2018). For RC structures, local weakening by material removal for seismic retrofit as a concept is discussed in a preliminary and general manner in FEMA (2000) with little detail. Severing of bottom longitudinal steel reinforcement has recently been explored in detail as a seismic retrofit method to protect exterior beam-column joints (Pampanin 2006; Viti et al. 2006; Kam and Pampanin 2008; Kam et al. 2009;), but cutting bottom bars cannot solve the problem associated with the contribution of slab for T-section beams under negative bending. The proposed BWFS method (Teng et al. 2013) represents an application/extension of the general selective local weakening approach to solve the slab contribution problem. Based on the concept of BWFS, the following three seismic retrofit techniques were proposed to enforce the strong column-weak beam hierarchy where necessary and/or appropriate (Teng et al. 2013):

1) The first technique, referred to as <u>the beam opening (BO) technique</u>, involves the creation of an opening on the web in each end region of a T-section beam adjacent to the beam-column joint, as shown in Fig. 4. The internal longitudinal steel reinforcement should be kept intact during the weakening process. If the opening is large enough, the flexural capacity of the T-section beam in negative

bending can be expected to reduce to a desired value. Local strengthening of regions adjacent to the opening (e.g. using FRP wraps and/or near-surface mounted FRP strips) is needed, particularly to ensure that the weakened beam still has an adequate shear resistance.

2) The second technique, referred to as the section reduction (SR) technique, involves the removal of concrete (and some of the longitudinal steel bars if necessary) from the bottom zone of the beam (i.e. the compression zone under negative bending) adjacent to the beam-column joint, as shown in Fig. 5. This method reduces the effective section height under negative bending and is expected to be highly effective in reducing the beam flexural capacity. The severing of some of the bottom longitudinal steel bars directly reduces the amount of longitudinal steel compression reinforcement under negative bending. Local strengthening of the region adjacent to the gap induced by material removal can also be implemented using FRP warps and/or near-surface mounted FRP strips.

3) The third technique, referred to as the slab slit (SS) technique, involves the separation of the slab in the corner region from each supporting beam by cutting a slit (including the severing of the steel bars crossing the slit) between them, as shown in Fig. 6. In this method, the path of stress transfer from the beam to the slab near the beam-column joint is weakened so that the contribution of a

cast-in-place slab to the beam flexural capacity in negative bending is substantially reduced or totally eliminated. Strengthening of the slab for its sagging moment capacity, as a result of the introduction of slits, can be easily achieved using FRP reinforcement if needed.

It should be noted that all the three techniques described above can be used in combination with column strengthening if necessary. While the BO method cannot be used together with the SR method, either of these two methods can be used in conjunction with the SS method to achieve a better weakening effect on the flexural capacity of the beam in negative bending.

For the BO technique, creating web openings in RC beams is not a new thing. In existing structures, if the passage of utility ducts/pipes is needed, cutting web openings in the beams is an appealing solution and has already been adopted in real projects (e.g. Mansur et al. 1999; Maaddawy and Sherif 2009; Maaddawy and Ariss 2012). This kind of web openings is usually located in the regions where the bending moment is small. If the web opening is moved to be located near the beam end, the two requirements can be met at the same time: one is to weaken the beam end to meet the strong column-weak beam hierarchy, and the other one is to meet the functional requirements such as electricity conduits, which is very attractive. The installation of a local strengthening system is needed following the creating of web opening to avoid shear failure of the weakened beam. Externally bonded FRP shear reinforcement has been shown by many researchers to be an effective method for enhancing the shear capacity of RC beams (Teng et al. 2002; Chen and Teng 2003a, b; Bousselham and Chaallal 2004; Al-Rousan and Issa 2016; Siddika et al. 2019). Nie et al. (2018) carried out an experimental study consisting of a total of 8 full-scale RC beams to investigate the effectiveness of the BO technique. The test results showed that a sizable web opening can effectively reduce the flexural capacity of a T-section beam, and the proposed FRP strengthening system can effectively avoid shear failure and ensure a ductile response of the beam.

For the SS technique, several numerical studies on the effectiveness of the SS technique have been conducted (Zhang et al. 2011; Wang et al. 2012; Zhang 2013; Xiao and Yin 2016), which are summarized in Table 10. All these studies indicated that joints/frames with slab slits can better achieve the strong column-weak beam mechanism than joints/frames without slab slits.

For the SR technique, the removal of the compressive concrete (and some of the steel bars) is expected to reduce the section flexural capacity significantly; this reduction can be easily estimated by a conventional section analysis, but the accuracy of such an approach does need some verification.

In addition to the three techniques proposed by Teng et al. (2013), Feng et al. (2017) proposed a novel method using kinked rebars in the beams for improving the seismic performance and progressive collapse resistance of RC frame structures. The kinked rebar has locally curved regions (usually near the inflection points in beams) which can be gradually straightened under tension (as shown in Fig. 7). Due to the lower initial yielding flexural capacity compared with that of a cross section reinforced with traditional straight bars, the beam section reinforced with kinked rebars will yield first when the RC frame is subjected to seismic loading, and thus the strong column-weak beam hierarchy can be realized (Feng et al. 2017; Qiang et al. 2019). Although this method was originally proposed for new construction, the concept has the potential to be adopted in the BWFS method for existing structures. The feasibility and effectiveness of kinked rebars in reducing the flexural capacity of the beam is worth further investigations.

To summarize, there are mainly two ways for the seismic retrofit of RC frames which violate the strong column-weak beam hierarchy: column strengthening and beam weakening. Strengthening techniques for RC columns mainly include concrete jacketing, steel jacketing and FRP jacketing. The former two techniques may lead to increases in mass and/or stiffness and then increases in seismic forces. FRP jacketing has been widely used in recent years as a simple but effective method for column strengthening, but the strength enhancement due to FRP jacketing may be small, especially when FRP jacketing is applied to confine non-circular columns. Another problem of column strengthening is that even when column strengthening can be sufficient, the location of failure may simply shift from column ends to the foundation and/or beam-column joints. Therefore, column strengthening alone is often not sufficient enough to change the strength hierarchy. It always needs to be combined with joint strengthening which is relatively complex. Weakening techniques for RC beams mainly include severing of bottom longitudinal steel reinforcement and the BWFS method proposed by Teng et al. (2013). Cutting bottom bars can reduce the positive flexural strength of RC beam, but it cannot solve the problem associated with the contribution of slab for T-section beams under negative bending. The BWFS method proposed by Teng et al. (2013) can reduce both the positive and negative flexural strengths of the RC beam with a cast-in-place floor slab. It should be noted that, however, additional measures need to be taken to support the RC beam/floor slab when the weakening process is in progress, in order to ensure the safety of the weakened beam.

7. CONCLUDING REMARKS

This paper has provided a review of the existing knowledge on the strong column-weak beam design philosophy, covering the concept of strong column-weak beam design, factors affecting the accurate calculation of the flexural capacity of beams $\sum M_B$, the determination of the column-to-beam flexural strength ratio η_C, the development of design provisions for the strong column-weak beam design philosophy, the current situation of the implementation of the strong column-weak beam design philosophy in structural design and seismic retrofit of existing RC frames. Based on the review and discussions presented in this paper, the following conclusions can be drawn:

1) The strong column-weak beam hierarchy has been widely adopted as one of the main design requirements in the seismic design of RC frame structures, in order to realize the beam-sway mechanism for RC frame structures subjected to seismic loading;
2) To achieve the strong column-weak beam hierarchy for an RC frame, the relationship $\sum M_C = \eta_C \sum M_B$ should be satisfied, where $\sum M_C$ and $\sum M_B$ are the sums of the flexural capacities of the columns and the beams framing into the joint respectively, and η_C is the column-to-beam flexural strength ratio and should be larger than 1.0;
3) The main factors which can lead to under-estimation of the flexural capacity of RC beams or degradation of the ductile performance of RC beams include the neglect of the capacity contribution from cast-in-place floor slabs, possibly over-reinforced beam ends and the effect of infill walls and gravity loads. The existence of a cast-in-place floor slab can significantly enhance the stiffness and strength of the beam supporting it; infill walls may significantly alter the failure mechanism of an RC frames; over-reinforced beam ends directly increase the flexural capacity of the beam at the ends; and the gravity loads may change the distribution of plastic hinges in the RC beams. Therefore, in the design of RC frame structures, the effect of the above factors on the flexural capacity of RC beams should be properly considered;

4) For the effect of cast-in-place floor slabs on the behaviour of RC frames, more experimental studies on spatial RC frames with cast-in-place floor slabs need to be conducted; the accuracies of the relevant numerical studies need to be verified using test results; the suggested effective flange widths vary largely among different studies, and a widely accepted effective flange width has not been established, which can be a future study topic;
5) For the numerical studies on the effect of infill walls on the behaviour of RC frames, the modeling methods of infill walls used by different researchers are quite different from each other and a widely accepted modeling method of infill walls has not been established, indicating that more relevant studies are needed;
6) A proper value of the column-to-beam flexural strength ratio η_C is very important to achieve the strong column-weak beam mechanism in RC frames. By now, there have been a large number of studies on the determination of η_C, and the suggested values of η_C by the researchers from different countries (e.g. China, United States and India) are mostly much larger than 1.0. However, there is a large variation among the suggested values of η_C by different researchers. More systemic studies (i.e. considering the effects of number and heights of storeys, number and lengths of spans, vertical loads etc.) need to be conducted to determine more reasonable and widely acceptable value of η_C;
7) Structures designed using old versions of design codes which did not adopt the strong column-weak beam design philosophy cannot or can hardly achieve the strong column-weak beam hierarchy. Although the newer design codes have

adopted the strong column-weak beam design philosophy, existing studies have indicated that the values of η_C stipulated in these codes are still insufficient to ensure the strong column-weak beam hierarchy. Therefore, it can be expected that a large number of existing RC frame structures violate this hierarchy requirement and need to be retrofitted;

8) Existing studies have indicated that column strengthening alone is often not sufficient to achieve the strong column-weak beam hierarchy. Three strengthening techniques based on the concept of beam-end weakening in combination with FRP strengthening (BWFS) were proposed by Prof Teng's group (Teng et al. 2013): (a) the beam opening (BO) technique; (b) the beam section reduction (SR) technique; and (c) the slab slit (SS) technique. The proposed techniques can be used alone or in combination with column strengthening; and

9) Although limited existing studies have proven the effectiveness of the BO technique and the SS technique, in-depth experimental, numerical and theoretical studies on the effectiveness of the BWFS method need to be conducted in the future.

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Table 1. Summary of numerical studies on the effect of cast-in-place floor slabs on the behaviour of RC frames

Source	Dimensions	FE Model	Modeling of floor slab	Analysis type	Software
Guan and Du (2005)	2-D	A 3-storey 3-span RC frame	T-section beam	Pushover analysis	SAP2000 (1998)
Lin et al. (2009)	3-D	Two 6-storey 3x9-span RC frames, one with slabs and one without slabs	Elastic shell element	Elastic-plastic time history analysis	MSC.Marc (2005)
Gao and Ma (2009)	3-D	Two 6-storey 4x4-span RC frames, one with slabs and one without slabs	Layered shell element	Pushover analysis	SAP2000 (1998)
Yang (2010)	3-D	Five 6-storey 4x4-span RC frames with different slab widths	Shell element	Pushover analysis	SAP2000 (1998)
Chen (2010)	3-D	Two 6-storey 6x3-span RC frames, one with slabs and one without slabs	Shell element	Elastic-plastic time history analysis	SAP2000 (1998)
Guo (2012)	3-D	Three 3-storey 3x4-span RC frames, one with slabs and slab reinforcement, one with slabs but without slab reinforcement, and one without slabs	Shell element	Pushover analysis	SAP2000 (1998)
Ning et al. (2016)	3-D	Two 2-storey 1x2-span RC frames, one with slabs and one without slabs	Shell element	Pushover analysis	ABAQUS (2006)

Table 2. Suggested effective flange widths

Source	Value of b_f	Example* (mm)	Applicable condition
Durrani and Zerbe (1987)	b_c + 2h	1300	Exterior joints
French and Moehle (1991)	min{l_0/4, b + 16t, s}	1500	Interior joints (γ = 1/50)
Li (1994)	b + 8t	1050	Interior joints
Jiang et al. (1994)	b + 12t	1450	Interior joints
Wu et al. (2002)	b + 12t	1450	Interior joints (γ =1.5%)
Wang et al. (2009)	b + 2t	450	Interior joints (γ =1/550)
Zhen et al. (2009)	min{b + 3.5h, l_0/3, s}	2000	Interior joints (γ =1/50)
	min{b + 1.5h, l_0/6, s}	1000	Exterior joints (γ =1/50)
Yang (2010)	b + (12~16)t	1450~1850	Interior joints
Sun (2010)	b + min{max(l_0/4, 2h), 1/2s}	1750	Interior joints (γ =1/50)
	b + min{max(l_0/5, 1.5h), 1/2s}	1450	Exterior joints (γ =1/50)
Qi et al. (2010)	b + min{l_0/4, 12t, s}	1450	Interior joints (γ =1/50)
	b + min{l_0/5, 8t, s}	1050	Exterior joints (γ =1/50)
He (2010)	b + 12t	1450	Interior joints (γ =1/50)
Ning et al. (2016)	b + 3.2h	1850	Interior joints (γ =1/50)

	b + 2.7h	1600	Exterior joints (γ =1/50)

Note: b_c=column width; b=beam width; h=beam height; l_0=effective span of beam; t=slab thickness; s=clear distance between two adjacent beams; γ = inter story drift angle.

*A typical case (b_c=300 mm; b=250 mm; h=500 mm; l_0=6 m; t=100 mm; s=3 m).

Table 3. Studies on the effect of infill walls on the behavior of RC frames

Source	Experimental study	Numerical study			
		FE Model	Modeling of infill walls	Analysis type	Software
Lin et al. (2009)	NA	Three 6-storey 3x9-span RC frames, one pure frame, one frame with floor slab, and one frame with both floor slab and infill walls	Elastic-plastic model and fracture constitutive model	Elastic-plastic time history analysis	MSC.Marc (2005)
Chen (2010)	NA	Four 6-storey 6x3-span RC frames, one pure frame, one frame with floor slab, one frame with both floor slab and infill walls, and one frame with both floor slab and infill walls except the ground floor	Shell element	Linear and non-linear time history analysis	SAP2000 (1998)
Xiong (2011)	NA	Two 5-storey 2-span RC frames, one pure frame and one frame with infill walls	Cross spring supporting model	Pushover analysis	SAP2000 (1998)
Shi (2012)	Two 1/4-scale 3-storey 2x2-span RC frames, one pure frame and one frame with infill walls	Two 1/4-scale 3-storey 2x2-span RC frames, one pure frame and one frame with infill walls	Equivalent diagonal strut model (double-strut model)	Elastic-plastic time history analysis	PERFORM-3D (2011)
Yuen and Kuang (2015)	NA	Six 2-storey 2-span RC frames, one pure frame, one frame with full infill walls, one frame with 2/3-storey-height infill walls, one frame	Solid element	Elastic-plastic time history analysis	ABAQUS (2006)

		with infill walls except the ground floor, one frame with full infill walls and window openings, and one frame with full infill walls and door openings			
Li et al. (2016)	Two 1/34-scale 2-storey 4-span RC frames, one pure frame and one frame with infill walls	Two 1/34-scale 2-storey 4-span RC frames, one pure frame and one frame with infill walls	Solid element	Elastic-plastic analysis	ABAQUS (2006)
Shan et al. (2016)	Two 1/34-scale 2-storey 4-span RC frames, one pure frame and one frame with infill walls	NA	NA	NA	NA
Fiore et al. (2016)	NA	Three 4-storey 5x3-span RC frames, one pure frame, one frame with infill walls, and one frame with infill walls except the ground floor	Equivalent diagonal strut model (single-strut model and double-strut model)	Pushover analysis	SAP2000 (1998)
Hadi et al. (2018)	Two full-scale 1-storey 1x1-span RC frames, one pure frame and one frame with infill walls	NA	NA	NA	NA
Mohammed and Güneyisi (2018)	NA	2-, 4-, 6- and 8-storey RC frames without infill walls or with four different infill wall arrangements	Equivalent diagonal strut model (single-strut model and triple-strut model)	Pushover analysis	SAP2000 (1998)

Table 4. Studies on the effect of over-reinforced beam ends on the behavior of RC frames

Source	**FE Model**	**Analysis type**	**Software**	**Remarks**
Lei (2002)	Three RC frames, frame A is a 6-storey 3-span frame of SG[a] 1 in SPI[b] 9 region in China, frame B is an 11-storey 3-span frame of SG 1 in SPI 8 region in China, frame C is an 8-storey 3-span frame of SG 2 in SPI 8 region in China	Elastic-plastic time history analysis	PL-AFJD (Yang 2000)	According to the Chinese code GB-50011 (2001), the flexural capacities of beams and columns were calculated based on the actual reinforcement for frames A and B and the design bending moment for frame C
Liu et al. (2004)	Two 6-storey 3-span RC frames, frame A of SG 1 in SPI 9 region in China and frame B of SG 2 in SPI 8 region in China	Elastic-plastic time history analysis	PL-AFJD (Yang 2000)	According to the Chinese code GB-50011 (2001), the flexural capacities of beams and columns were calculated based on the actual reinforcement for frame A and the design bending moment for frame B
Han et al. (2010)	Two RC frames, frame A didn't have over-reinforced beam end while frame B had beam ends which were over-reinforced by 10%	Pushover analysis	OpenSees (2009)	NA

Note: (a) SG= Seismic Grade; (b) SPI= Seismic Precautionary Intensity.

Table 5. Suggested values of column-to-beam flexural strength ratio η_C

Source	Calculation of flexural capacity of the beam: Reinforcement	Calculation of flexural capacity of the beam: Consideration of the effect of floor slabs	Value of η_C	Remarks
Xu et al. (1986)	Designed reinforcement	No	1.42 - 2.86	η_C of each joint in a tested frame
Dooley and Bracci (2001)	Actual reinforcement	Yes	2.0	For RC frames in the US
Wei et al. (2003)	Designed reinforcement	No	1.4-1.5	For RC frames of SG 2 in an SPI 8 region in China
Ma and Chen (2005)	Designed reinforcement	No	1.6	For RC frames in an SPI 8 region in China
Cai et al. (2007)	Actual reinforcement	Yes	2.0	For RC frames in an SPI 8 region in China
Wei et al. (2007)	Designed reinforcement	No	1.3	For RC frames of SG 3 in an SPI 7 region in China
	Actual reinforcement	No	1.0	For RC frames of SG 2 in an SPI 8 region in China
Xia (2009)	Designed reinforcement	No	1.4	For RC frames of SG 2 in an SPI 8 region in China
			1.3	For RC frames of SG 2 in an SPI 7 region in China
			1.2	For RC frames of SG 3 in an SPI 7 region in China
Tao (2010)	Designed reinforcement	No	1.7	For RC frames of SG 2 in China
Yang (2010)	Designed reinforcement	No	1.6-2.0	For RC frames of SG 2 in an SPI 8 region in China
Ye et al. (2010)	Actual reinforcement	Yes	2.0 (1.7)	For RC frames of SG 1 (the latter one is the suggested moderate value of the former one)
			1.7 (1.5)	For RC frames of SG 2 in China
			1.4 (1.3)	For RC frames of SG 3 in China
Sun (2010)	Designed reinforcement	No	1.8-2.0	For RC frames of SG 2 in China
Yang (2011)	Actual reinforcement	Yes	2.4	For RC frames of SG 1 in China
			2.1	For RC frames of SG 2 in China
			1.9	For RC frames of SG 3 in China
			1.6	For RC frames of SG 4 in China
Yang (2012)	Designed reinforcement	No	1.5	For RC frames of SG 3 in China
Sunitha et al. (2014)	Designed reinforcement	No	2.5-3.0	For RC frames in India
Sargar and Bhusari (2018)	Designed reinforcement	No	＞1.4	For RC frames in India

Table 6. Stipulations on the strong column-weak beam requirement of RC frames in NZS-3101

Time	Stipulation	Remarks
1982	$\Sigma M_C \geq (1.6\sim2.4)\Sigma M_B$	ΣM_C: sum of the moments at ideal strength in hinging columns at opposite faces of the joint, summed in the same vector sense, and related to the centre of the intersect-beam; ΣM_B: sum of the moments at ideal strength in non-yielding beams at opposite faces of the joint, summed in the same vector sense, and related to the centre of the intersecting beam. Slab reinforcement within an effective flange width d_f shall be assumed to contribute to M_B; $d_f = b + 8t$.[(a)]
2006	$\Sigma M_C = \omega\beta\Sigma M_B$ $\beta = 1.4 - \frac{\sum M'_o}{2.5\emptyset_{o,fy}\sum M'_n}$	ΣM_C: sum of nominal flexural strengths of the columns framing into that joint, evaluated at the faces of the joint; ΣM_B: sum of bending moments in beams sustained at the intersection of the beam and column centrelines when nominal moments act in the beams at the column faces. Slab reinforcement within an effective flange width d_f shall be assumed to contribute to M_B; $d_f = b + \min\{2h, 16t, 2s*h_{b1}/(h_{b1}+h_{b2}), l_0/4\}$[(a)], where h_{b1} is the depth of the beam being considered and h_{b2} is the depth of the adjacent beam; ω: appropriate dynamic magnification factor, not less than 1.3 and not more than 1.8; β: appropriate modification factor; $\sum M'_o$ and $\sum M'_n$ are the sums of the beam overstrength and nominal strength moments respectively, acting at the column faces of the beam column joint being considered; $\emptyset_{o,fy}$ = 1.25 for Grade 300 reinforcement; = 1.35 for Grade 500 reinforcement.

Note: (a) The notation follows that in Table 2.

Table 7. Stipulations on the strong column-weak beam requirement of RC frames in ACI 318

Time	Stipulation	Remarks
1983	$\Sigma M_C \geq (6/5)\Sigma M_B$	ΣM_C: sum of moments, at the center of the joint, corresponding to the design flexural strength of the columns framing into that joint; ΣM_B: sum of moments, at the center of the joint, corresponding to the design flexural strengths of the girders framing into that joint.
2002	$\Sigma M_C \geq (6/5)\Sigma M_B$	ΣM_C: sum of moments at the face of the joint corresponding to the nominal flexural strength of the columns framing into that joint; ΣM_B: sum of moments at the face of the joint corresponding to the nominal flexural strength of the girders framing into that joint. In T-beam construction, where the slab is in tension under moments at the face of the joint, slab reinforcement within an effective flange width d_f shall be assumed to contribute to flexural strength if the slab reinforcement is developed at the critical section for flexure; $d_f = \min\{l_0/4, b + s, b + 16t\}$.[a]
2005	$\Sigma M_C \geq (6/5)\Sigma M_B$	ΣM_C: sum of nominal flexural strengths of the columns framing into that joint, evaluated at the faces of the joint; ΣM_B: sum of nominal flexural strengths of the beams framing into that joint, evaluated at the faces of the joint. In T-beam construction, where the slab is in tension under moments at the face of the joint, slab reinforcement within an effective flange width d_f shall be assumed to contribute to M_B if the slab reinforcement is developed at the critical section for flexure; $d_f = \min\{l_0/4, b + s, b + 16t\}$.[a]
2019	$\Sigma M_C \geq (6/5)\Sigma M_B$	Same as above except the stipulation for d_f. $d_f = b + \min\{l_0/4, s, 16t\}$.[a]

Note: (a) The notation follows that in Table 2.

Table 8. Stipulations on the strong column-weak beam requirement of RC frames in Eurocode 8

Time	Stipulation	Remarks
1995	$\Sigma M_C \geq 1.3 \Sigma M_B$	ΣM_C: the sum of design values of the flexural capacity of the columns framing into a joint; ΣM_B: sum of design values of the flexural capacity of the beams framing into a joint; slab reinforcement parallel to the beam and within the effective flange width d_f should be assumed to contribute to the beam flexural capacities and taken into account for the calculation of ΣM_B, if it is anchored beyond the beam section at the face of the joint; $d_f = b_c + 8t$. [(a)]
2004	Same as above.	Same as above.

Note:

(a) The notation follows that in Table 2.

Table 9. Stipulations on the strong column-weak beam requirement of RC frames in Chinese design codes

Time	Code	Stipulation				Remarks
		SG 1	SG 2	SG 3	SG 4	
1989	GBJ11-89	$\Sigma M_C = 1.1 \Sigma M_{Bua}$ or $\Sigma M_C = 1.1 \lambda_j \Sigma M_B$	$\Sigma M_C = 1.1 \Sigma M_B$	NA		ΣM_C: sum of design flexural capacities of the columns framing into a joint; ΣM_B: sum of design flexural capacities of the beams framing into a joint; λ_j: amplified coefficient due to over-reinforcement, 1.1 can be used; ΣM_{Bua}: sum of flexural capacities of the beams framing into a joint calculated based on the actual reinforcement and the standard strength of materials, the effect of cast-in-place floor slab is not considered.
2001	GB-50011	$\Sigma M_C = 1.4 \Sigma M_B$ or $\Sigma M_C = 1.2 \Sigma M_{Bua}$	$\Sigma M_C = 1.2 \Sigma M_B$	$\Sigma M_C = 1.1 \Sigma M_B$	NA	Same as above.
2010	GB-50011	$\Sigma M_C = 1.7 \Sigma M_B$ or $\Sigma M_C = 1.2 \Sigma M_{Bua}$	$\Sigma M_C = 1.5 \Sigma M_B$	$\Sigma M_C = 1.3 \Sigma M_B$	$\Sigma M_C = 1.2 \Sigma M_B$	Same as above except ΣM_{Bua}. When there is cast-in-place floor slab, actual reinforcement at beam end should include reinforcement in the slab within effective flange width d_f. $d_f = \min\{l_0/3, b + s, b + 12t\}$.[(a)]

Note: (a) The notation follows that in Table 2.

Table 10. Summary of studies on the effectiveness of the slab slit (SS) technique

Source	**FE Model**	**Modeling of floor slab**	**Analysis type**	**Software**
Zhang et al. (2011)	Three RC joints with a floor slab, joint A without slits, joint B with slits whose length was 200 mm, joint C with slits whose length was 300 mm	Solid element	Quasi-static analysis	ADINA (2007)
Wang et al. (2012)	Two 1/2 scale 6-storey 2-span RC frames, frame A without slits and frame B with slits whose length was 200 mm	NA	Linear and non-linear time history analysis	ADINA (2007)
Zhang (2013)	Three 6-storey 2x3-span RC frames, normal frame A, frame B with slits whose length was 200 mm and frame C whose columns were strengthened with FRP	Solid element	Elastic-plastic time history analysis	ABAQUS (2006)
	Two 5-storey 2x11-span RC frames, normal frame D and frame E with slits whose length was 200 mm	Solid element	Elastic-plastic time history analysis	SAP2000 (1998)
Xiao and Yin (2016)	Six RC joints with a floor slab, joint A without slits, joint B with slits whose length was 360 mm, joint C with slits whose length was 480 mm, joint D with slits whose length was 600 mm, joint E with slits whose length was 720 mm, joint F with slits whose length was 900 mm	Shell element	Quasi-static analysis	MSC.Marc (2005)

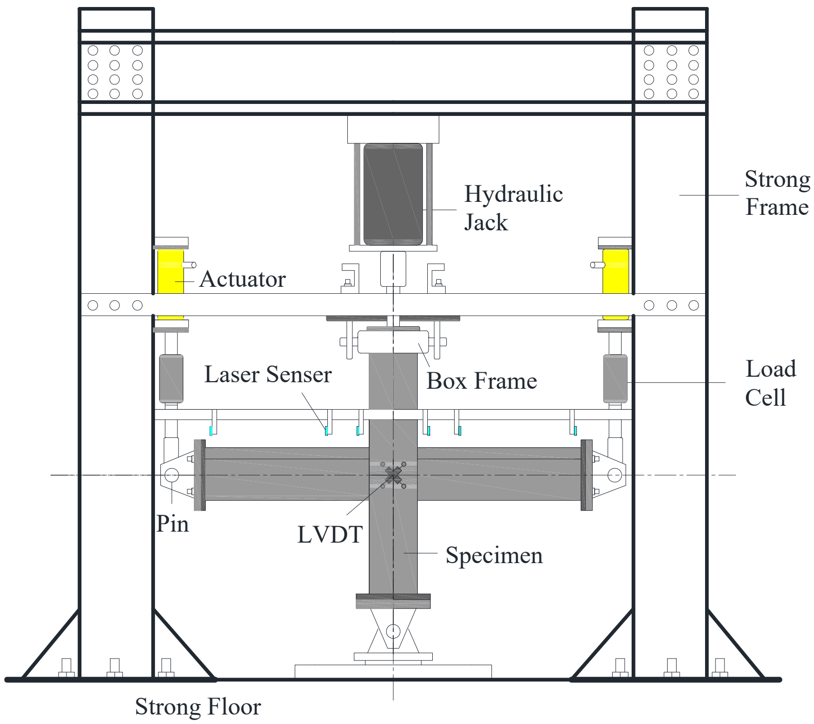

Figure 1. Typical test setup of RC joints with cast-in-place floor slabs

Figure 2. Failure at column ends

(Courtesy of Prof. P. Feng, Tsinghua University, China)

Figure 3. Failure at beam ends

(Courtesy of Dr. D.H. Jing, Southeast University, China)

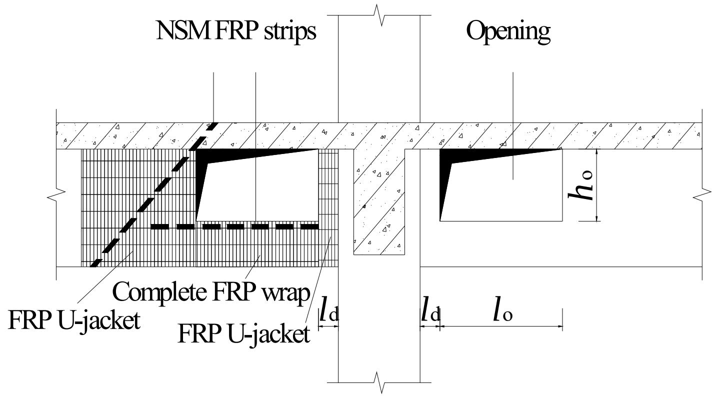

Figure 4. Beam opening (BO) technique

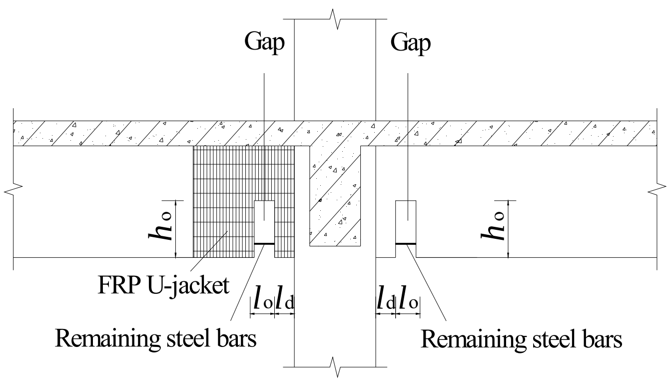

Figure 5. Section reduction (SR) technique

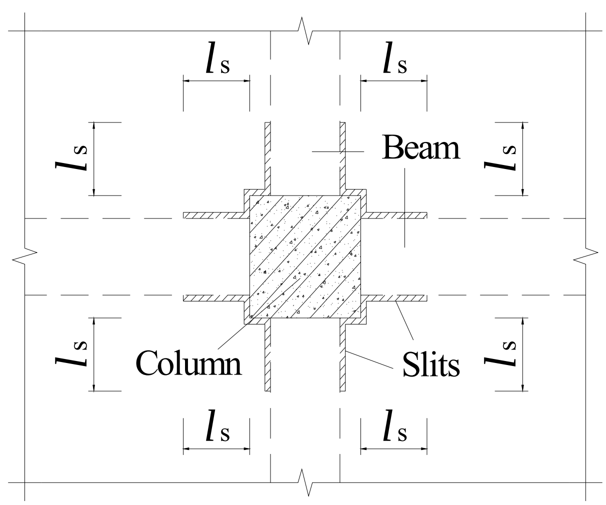

Figure 6. Slab slit (SS) technique

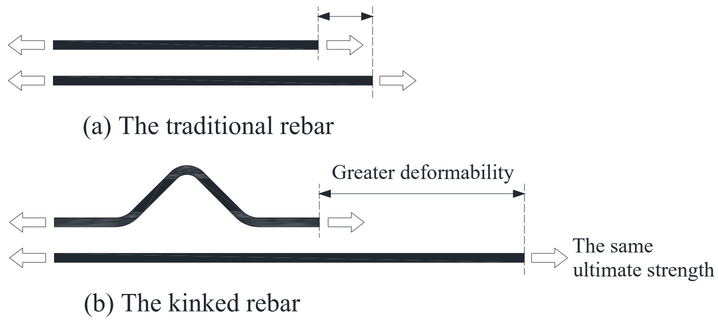

Figure 7. Diagram of kinked rebar