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Design Requirements of Reinforced Concrete Beam-column Joints in International Codes and ECP-RC-2018

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Abstract: *During past earthquakes, the behavior of framed structures was poor due to shear failure, or buckling of column bars in the joint zone. The results of recent researches were carried out to investigate the complex mechanism of the joints, and implemented into code recommendations. Therefore, the aim of this research is investigating the recommendations of international codes regarding design and detailing aspects of beam column joints, as well as a comparison of the Egyptian code (2018), ECP-RC-18 [1], provisions to ACI318-14 [2], ACI352R-02 [3] is provided. All three codes aim to satisfy the bond and shear requirements within the joint. The codes state high importance for design and detailing of joints especially in seismic zones through providing adequate anchorage of longitudinal bars and confinement of core concrete in resisting shear. Significant factors influencing the design of beam-column joints are identified and the effect of their variations on design parameters is discussed. It was concluded that, the required length, in ECP-RC-18, for controlling slippage of beam and column bars that passing through the joint need to be increased by 20 to 40%. For discontinuous columns, the vertical U- shape transverse stirrups should be considered due to its importance for improving joint performance during earthquakes. Added to that, the required area of confinement reinforcement in the joint for spiral hoops, dealing with eccentric beam framed into the joint, modifying the factor of confinement coefficient to match ACI code are needed to be considered in ECP-RC-18. Finally, it is preferable to redesign and strengthen the joints in buildings that located in earthquake zones, which were built before the development of current Egyptian design guidelines.*

Keywords: *beam-column joint, design requirement of joint, Shear reinforcement of joint, international codes provision, joint failure mechanism, ACI 318-14, ACI 352R-02, ECP- RC- 2018.*

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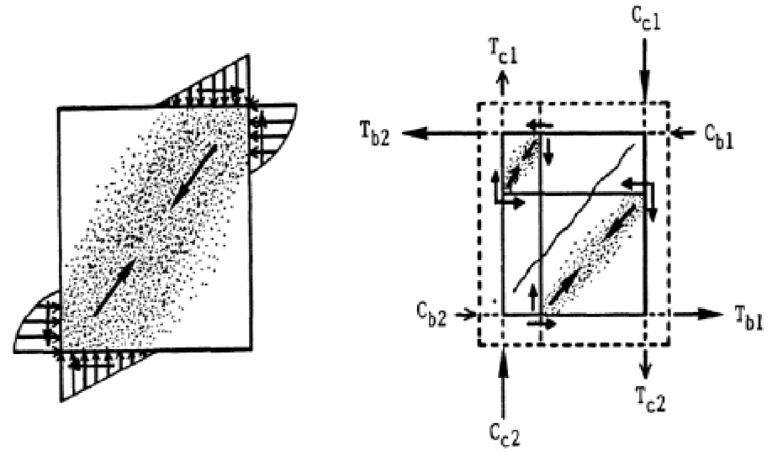
I. Introduction

One of the basic assumptions of the frame analysis is that the joints are strong enough to sustain the forces (moments, axial and shear forces) generated by the loading, and to transfer the forces from one structural member to another (beams to columns, in most of the cases). It is also assumed that all the joints are rigid, and the members meeting at a joint deform (rotate) by the same angle. Hence, it is clear that unless the joints are designed to sustain these forces and deformations, the performance of structures will not be satisfactory under all the loading conditions, especially under seismic conditions. However, the catastrophic failures reported in the past earthquakes, especially during the past several earthquakes in many places all over the world, were attributed to beam-column joints [4~14], as shown in Fig. (1). Therefore, in recent codes, the design of beam-column joint under seismic earthquake conditions became
(a) the gravity load is sustained by the joint.
(b) a large ductility and energy dissipation is hard to achieve in the joint.
(c) a joint is difficult to repair after an earthquake.

However, an excessive complication of reinforcement detailing should be equally avoided to insure good workmanship and construction. Therefore, joint shear failure and a significant beam bar slippage within a joint should be prevented up to an expected structural deformation. The shear resisting mechanisms of beam-column joint was through two categories, as shown in Fig.(2), Y. C. Choi, et al. (2017) & Uma, S.R. et al. (2006) [4,5]. The first one is the diagonal strut mechanism, i.e., the diagonal compression strut is formed along the main diagonal of a joint panel by the resultant of the horizontal and vertical Compression stresses and shear stresses acting on the concrete at the beam and column critical sections. The latter one is the truss mechanism, which is formed with uniformly distributed diagonal compression stresses, tensile stresses in the vertical and horizontal reinforcement and the bond stresses acting along the beam and column exterior bars.

Figure no 1: Beam-column joints failures in Izmit earthquake in Turkey



a) Diagonal strut mechanism b) Truss mechanism

Figure no2: Shear transfer mechanism in joint, [4, 5].

Defects in design and construction of joints

Some of the incorrect detailing are summarized as follows, Subramanian, (2015) [6]:

- Incorrect details of top beam reinforcement into the beam-column joint for anchorage. The top bars are bent in upwards direction instead of downwards. This leads to preventing diagonal strut formation in the joint and may cause diagonal cracking, as well as, shear failure of the joint.
- Inadequate anchorage of beam bars into the beam-column joint
- Poor quality of casting concrete at the joint zone.
- Shear reinforcement is not provided in the joints.
- Passing the extreme bars of the beam reinforcement outside the column bars except in a wide beam case.

Therefore, recent codes, ACI318-14, ACI352R-02, ECP-RC-18 [1,2,3], give great attention to joint design and reinforcing details, whereas, the joints are critical zones for transfer forces and moments effectively between the connected elements, i.e., beams and columns.

Requirements of beam-column joints

When buildings are subjected to lateral loads (earthquake, and/or wind) plastic hinges will be formed at the ends of the members, due to generated bending moments. When numerous of plastic hinges informed in the frame, it will fail in a mechanism. Therefore, the basic requirement of design is that the joint must be stronger than the adjoining hinging members. It is important in the design stage that the joint size is adequate otherwise the adjoining member size may need to be modified to satisfy the joint strength or anchorage requirements. The joint reinforcement detailing, (flexural or transverse), is importance to ensure having the full strength of reinforcing bars. Based on both recent codes and Paulay& Priestley (1992) [7], the essential requirements for the satisfactory performance of a joint in an RC structure, during earthquakes, can be summarized in the following steps:

a. The minimum dimensions for the joint based on anchorage requirements of beams longitudinal bars.
b. Achieve the week beam-strong column yielding mechanism through satisfying adequate flexural strength of columns
c. The design shear force acting on the joint through equilibrium between the flexural strength of opposite beams and the corresponding forces in columns.
d. The effective joint shear area based on the adjoining member dimensions.
e. Be sure that the shear stress of the joint is less than the nominal shear stress. It is checked through both the compressive strength and diagonal tensile strength of concrete.
f. Design shear reinforcement, i.e., both transverse and vertical reinforcements.
g. Provide the adequate anchorage length of the reinforcement in the joint zone.

The above listed points will be discussed in detail with respect to international codes as well as Egyptian code provisions.

II. Joints of special moment frames

The formation of plastic hinges at the ends of beams may result in significant shear force reversals in beam-column joints. Joint shear can be determined by computing the internal forces acting on the joint while assuming that the tension beam reinforcement anchored into the joint develops 1.25 f_y.

Types of Joints

Typical beam-column joints are defined as Type 1 and Type 2 joints, as per ACI 352R-02 [3] and ECP-RC-18 [1]:

- Type 1 Joints: these joints have members designed to satisfy strength requirements, without significant inelastic deformation. These are non-seismic joints.
- Type 2 Joints: these joints have members that are required to dissipate energy through reversals of deformation into the inelastic range. These are seismic joints.

But in ACI 318-14 [2], used another two categories, the first one is for beam-column joint that transfer moment to column shall satisfy detailing provision in chapter (15). The other category is for beam-column joint within special moment frames and in frames that not designed as part of seismic force resisting system in structures assigned to seismic design categories D, E, and F shall satisfy chapter (18).

III. Joint shear strength

The joint shear produces diagonal tension and compression reversals which may be critical for premature diagonal tension or compression failures, unless properly reinforced. The joint shear may especially be critical in edge and corner joints, which are not confined by the adjoining beams on all four faces. A member that frames into a joint face is considered to provide confinement to the joint if at least ¾ of the face of the joint is covered by the framing member. The shear capacity of beam-column joints in special moment resisting frames can be computed by the following expressions given in Sec. 18.8.4, Table (18.8.4.1:Nominal joint shear strength) of ACI 318-14 [2].

a) For joints confined on all four faces:

$$V_n \leq 1.7\sqrt{f_c'}A_j \qquad (1a)$$

b) For joints confined on three or two opposite faces:

$$V_n \leq 1.2\sqrt{f_c'}A_j \qquad (1b)$$

c) For other joints:

$$V_n \leq 1.0\sqrt{f_c'}A_j \qquad (1c)$$

Where, Aj is the effective joint cross-sectional area as defined in sec. (18.8.4.3) of ACI318-14 [2] and sec. (6-6-2) of ECP-RC-18 [1], as shown in Fig. (3).A_j equals the column depth "h_c", (h_c is column dimension in the direction of joint shear) times the effective width of the column, which is equal to the column width except where the beams frame into a wider column. In this case, the joint width shall not exceed the smallest values of the following:

- Beam width plus joint depth ($b_b + h_c$)
- Twice the smaller perpendicular distance from longitudinal axis of beam to column side, $2\times [(b_b/2 + x)]$

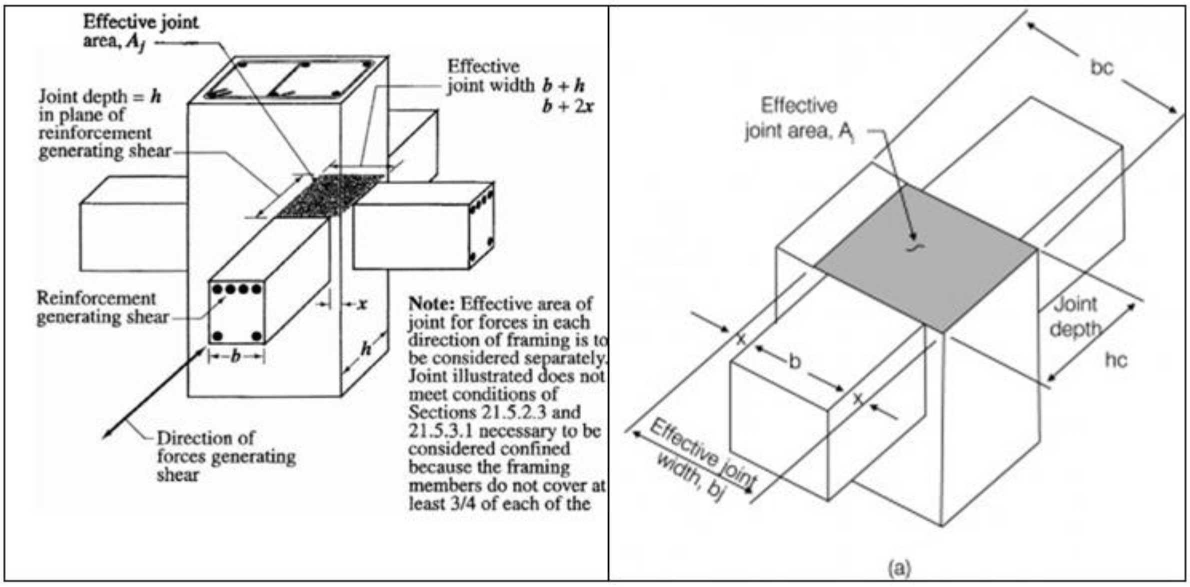


Figure no 3: Definition of effective joint area $A_{J,}$ [1&2].

Table no 1: Values of γ for beam-column connections, [3]

Symbol	Classification	Connection type	
		1	2
	Joint with a continuous column		
A1	Joint confined on all four vertical faces	24	20
A2	Joint confined on three vertical faces/or on two opposite vertical faces	20	15
A3	Other Cases	15	12
Joint with discontinuous column, (at roof as example)			
B1	Joint confined on all four vertical faces	20	15
B2	Joint confined on three vertical faces/ or on two opposite vertical faces	15	12
B3	Other Cases	12	8

The beam-column joint is defined as the portion of the column within the depth of the deepest beam that frames into the column, ACI 352R-02 [3]. The types of beam-column joints in a moment resistant frame, can be classified as (a) interior joint, (four beams frame into the vertical faces of a column), (b) exterior joint, (one beam frames into the vertical face of a column and two more beams frame into the column in the perpendicular direction), and (c) corner joint, (one in which beams frame into two adjacent vertical faces of a column). In a roof joint (knee joint), the columns are not extent above the joint, whereas in a floor joint, the columns extend above the joint. Based on joint classification the constant γ is given in Table (1). From Table (1), the member that frames into a joint face is considered to provide confinement to the joint if:

- The horizontal frame member cover at least ¾ of column's width.
- The total depth of confining member is not less than ¾ the total depth of the deepest member framing into the joint. (Noted that, this condition is not mentioned in ECP-RC-18 [1]).

It should be noted that:

- Both ACI 352R-02, and ECP-RC-18 [1 & 3], thenominal joint shear strength is calculated based on equation ($v_n = 0.083\ \gamma \sqrt{f'_c} b_j h_c$)& ($v_u = k_j b_j h_c \sqrt{(f_{cu}/\gamma_c)}$, where k_j= 0.083 γ)respectively.
- ACI 318-14 [2] is not distinguished betweenJoint with a continuous column and Joint with discontinuous column in calculating nominal joint shear strength.
- ACI 318-14 consider the values of γ that mentioned in Table (1), type 2. For example, the constant 1.7 in Eq.(1a) is generated from multiplying ($0.083\times20= 1.66\approx1.70$).
- The factor of k_j in for cases (A1&2-type 2 and B1&2 type 1) is overestimated by 7%. This leads to increase the values of allowable maximum shear force of the joint than values estimated in ACI codes [2 & 3].
- Example: given Cube concrete compressive strength f_{cu} = 30MPa, and the corresponding concrete cylinder $f'_c = 0.75\times30 = 22.5$ MPa, material strength reduction factor $\gamma c = 1.50$, φ: is the strength reduction factor equal to 0.85. Hence, constant value of Vu [2 &3] and Vu [1] are as follows:

$$\text{Constant of } V_{u(ACI)} = \varphi \times 0.083 \times \gamma \quad \text{and} \quad V_{u(ECP-RC18)} = k_j \times \sqrt{\frac{4/3}{\gamma_c}}$$

$$\therefore V_{u(ACI)} = 0.07055\,\gamma \qquad and \qquad V_{u(ECP-RC18)} = 0.94281 k_j$$

Table no 2: Comparison between [ACI 352R-02] and [ECP-RC18] values of ultimate design shear forcefor joints

Symbol	ACI 352R-02 [3]		ECP-RC-18[1]		Ratio (ECP-RC)/(ACI)
	γ	Constant	k_j	Constant	
A1	20	1.411	1.60	1.508	+6.87%
A2	15	1.058	1.20	1.1313	+6.93%
A3	12	0.8466	0.9	0.8485	0.22%

The design and detailing procedures as per International codes and Egyptian code, [1, 2, 3] are summarized in the following sections.

IV. Joint reinforcement

The column confinement reinforcement provided at the ends of columns should continue into the beam-column joint if the joint is not confined by the framing beams on all four faces. For interior joints, with attached beams externally confining the joint on all four faces, the spacing of joint reinforcement can be relaxed to 150 mm.

The performance of framed structures depends not only upon the individual structural elements but also upon the integrity of the joints. In most of the cases, joints of framed structures are subjected to the most critical loading under seismic conditions. However, despite the significance of the joints in sustaining large deformations and forces during earthquakes, specific guidelines are not explicitly included in codes of practice for their design and detailing until recently. Only some provisions have been included based on ACI 318-14[2] and ACI 352R-02 [3] codes.

While considerable attention is devoted to the design of individual elements (slabs, beams and columns), no conscious efforts are made to design joints in the absence of suitable guidelines. It appears that the integrity and strength of such joints are assumed to be satisfied by just anchoring the beam reinforcement in the joints.

One of the basic assumptions of the frame analysis is that the joints are strong enough to sustain the forces (moments, axial and shear forces) generated by the loading, and to transfer the forces from one structural member to another (beams to columns, in most of the cases). It is also assumed that all the joints are rigid, and the members meeting at a joint deform (rotate) by the same angle. Hence, it is clear that unless the joints are designed to sustain these forces and deformations, the performance of structures will not be satisfactory under all the loading conditions, especially under seismic conditions. Post-earthquake analyses of structures, show that the distress in the joint region is the most frequent cause of failure, rather than the failure of the connected elements ,Rai and Seth (2002) [8].Analytical models which simulate the response of reinforced concrete interior beam-column joints have been developed and implemented into Open Sees website (www.opensees.berkeley.edu), but they are complicated and not suitable for design office use.

Beam-Column Joints (transverse reinforcements) in frames

The beam-column joint in a multi-storey frame, transfers the loads and moments at the ends of the beams into the columns. For a four-member connection as shown in Fig.(4a), if the two beam moments are in equilibrium with one another then no additional reinforcement is required.

Uma, S.R.et al. (2005) [9], In the case of lateral loading like seismic loading, the equilibrating forces from beams and columns, as shown in Fig. (4b) develop diagonal tensile and compressive stresses within the joint. Cracks develop perpendicular to the tension diagonal A-B in the joint and at the faces of the joint where the beam passes through the joint. Concrete being weak in tension, therefore, transverse reinforcements have to be provided, across the plane of failure, to resist the diagonal tensile forces.

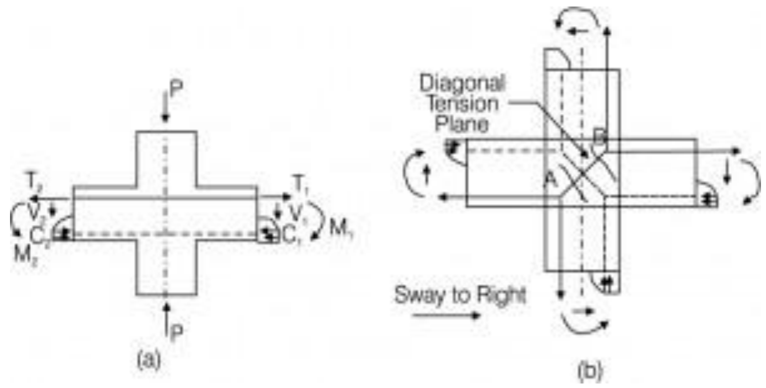


Figure no 4: Seismic loads develop diagonal tensile and compressive stresses within the joint, [9]**.**

V. Failure modes of beam-column Joints

As the joint zone area is small relative to the member sizes, it is essential to consider localized stress distribution within the joints. A simplified force system may be adopted in designing beam - column connections. The calculation of joint reinforcement is based on the assumption that both steel bars and concrete reach the design yield stress and the design compressive stress respectively. The failure due to local bearing, bond, and insufficient anchorage should be prevented within the joints, through proper design and detailing practices, Uma, S.R.et al. (2005) [9].The principal mechanisms of failure of a beam-column joint are:

- Shear failure within the joint.
- Anchorage failure of bars, if anchored within the joint, i.e., exterior or corner joint.
- Bond failure of beam or column bars passing through the joint.

As mentioned above, the joint has to be designed based on the fundamental concept that failure should not occur within the joint; that is, weak beam strong column phenomena.

Joint Shear and anchorage

Joint shear is a critical check and will govern the size of the columns of moment resisting frames. To illustrate the procedure, consider the column bounded by two beams. For ductile behavior, it is assumed that the beams framing into the column will develop plastic hinges at the ends and develop their probable moment of resistance (M_p) at the column faces. This action determines the demands on the column and the beam column joint.

Hanson and Connor (1967) [10], first suggested a quantitative definition of RC joint shear, namely that it could be determined from a free body diagram at mid-height of a joint panel.Fig.(5) is the free body diagram of the joint for calculation of column shear, V_c. It is made by cutting through the beam plastic hinges on both sides of the column and cutting through the column one-half storey height above and below the joint. In this figure, subscripts b_1 and b_2 refer to beams 1 and 2 on opposite sides of the joint, and $V_{b1}$and V_{b2} are the shears in the beams at the joint face corresponding to development of M_p at both ends of the beam. For a typical storey, it is sufficiently accurate to assume that the point of contra flexure is at the mid-height of column.

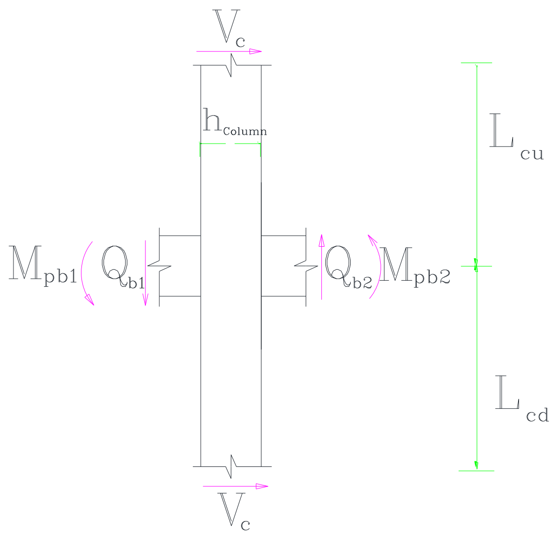


Figure no 5: The column shear is based on flexural capacities of beams, [10] Having found the column shear, V_c, the design horizontal joint shear V_{uj} can be obtained by considering the equilibrium of horizontal forces acting on the free body diagram of the joint shear as shown in Fig. (6),ACI352R-02 [3] . Assuming the beam to have zero axial load, the flexural compression force in the beam on one side of the joint may be taken equal to the flexural tension force on the same side of the joint, Moehle, et al. (2008) [11]. Thus the joint shear, V_{uj} is given by:

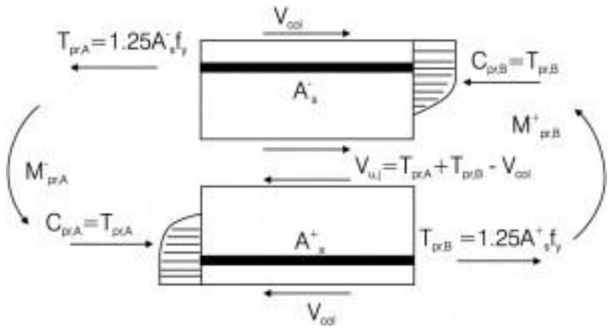


Figure no 6: Shear forces acting on the joint, [3, 11]

$$V_c = \frac{(M_{pb1} + M_{pb2})}{(l_{cu} + l_{cd})/2} \qquad (2)$$

$$V_{uj} = T_{pb1} + C_{pb2} - V_c \qquad (3a)$$

$$C_{pb2} = T_{pb2}\,(Zero\ axial\ load) \qquad (3b)$$

$$V_{uj} = T_{pb1} + T_{pb2} - V_c = \alpha\, f_y(A_{sb1} + A_{sb2}) - V_c \qquad (3c)$$

For an external joint, where the joint has beam on one side of the joint only, the above equation is written as:

$$V_{uj} = T_{pb1} - V_c = \alpha\, A_{sb1} f_y - V_c \qquad (4)$$

As mentioned in (ACI 352R-02), the factor α is a stress multiplier and α = 1 for Type 1 joints, where only limited ductility is required and α = 1.25 for type-2 joints, which require considerable ductility. The value of α = 1.25 is intended to account for:

(a) the actual yield stress of a typical reinforcing bar being commonly 10 to 25% higher than the nominal value.

(b) the effect of strain hardening at higher strain.

Since, the presence of a slab have a significant effect on the performance of Type 2 connections, The ACI 352R-02R-02 recommends including the longitudinal reinforcement in the slab within the effective width, in calculating the joint shear force. The effective width is as given in both ACI 318:14,and ECP-RC-18, Fig. (3).

The nominal shear strength of the joint V_{nj} should be at least equal to the required strength V_{uj}. Thus,

$$\varphi\, V_n \geq V_u \qquad (5a)$$

$$V_n = 0.083\,\gamma\,\sqrt{f_c'}\,b_j\,h_c \qquad (5b)$$

Where:

As shown in Fig.(3), b_j: is the effective width of the joint, and h_c:is the effective depth of the joint. φ: is the strength reduction factor equal to 0.85, and γ: is the coefficient of configuration and confinement of the joint provided by the beams, as mentioned in Table (1).

From equation (5a), the size of the column "h_c" should be increased, if the required strength V_{uj} is higher than the nominal shear strength of the joint V_{nj}.

Comment 1: It should be noted that, the effect of applied axial load in column is not considered in predicting the nominal shear strength of the joint, as mentioned in recent research, [Kim et al,. 2008].

Comment 2: In ACI 352R-02, if the eccentricity between the beam centerline and the column centroid exceeds1/8th of the column width, as shown in Fig. (7), the nominal shear strength V_{nj} of eccentric beam-column connection is reduced. This reduction referred to reduce the magnitude of coefficient m from 0.5 to 0.3 in calculating the effective joint width, i.e., bj = bb + ∑(m hc/2).

Comment 3: The ECP-RC-18, does not state any guidelines for designers to deal with eccentric beam framed into the joint.

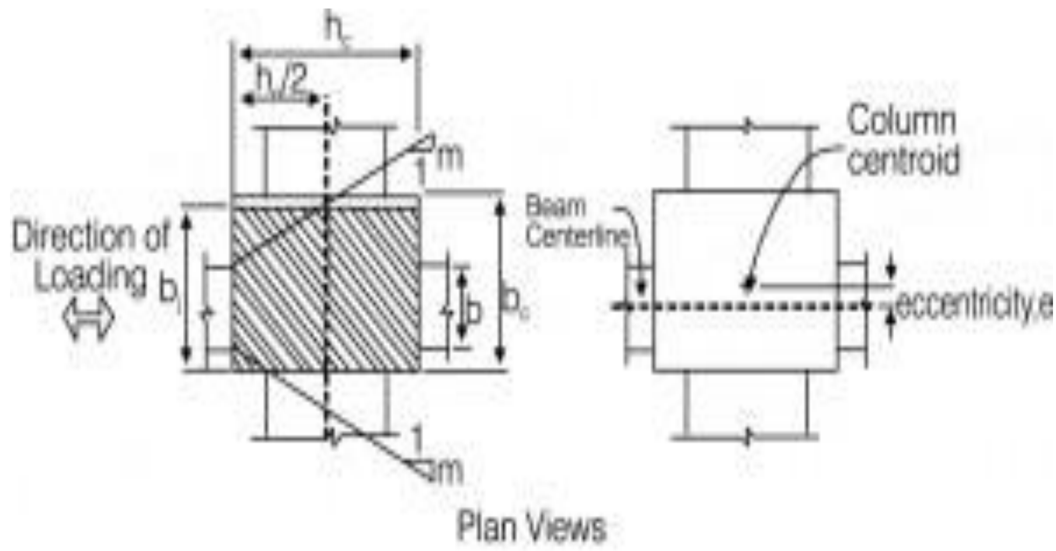


Figure no 7: Effective joint width for unequal beam and column width, [3]

Design of shear reinforcement

Paulay et al. (1992) [7], proposed shear transfer mechanisms of the joint referred to diagonal strut mechanism and truss mechanism, as shown in Fig. (2). It was considered that, the strength of the diagonal strut controls the joint strength before cracking. When the joint shear becomes large, diagonal cracking occurs in the joint zone and the joint reinforcements start to resist the shear forces. Finally the joint fails by crushing of the concrete in the joint zone.

Both ACI 318-14 [2] and ACI 352R-02 [3] assume that, the bond deterioration of the reinforcing bars in the joint zone occurs first then, the internal shear forces are resisted only by the diagonal compressive strut of concrete. Thus, the role of transverse reinforcement is only to confine the core concrete. Hwang et al. 2005 [13], mentioned that, the real behavior of the structure may be due to the combination of the diagonal strut and the truss mechanisms with the bond deterioration of longitudinal reinforcement to a certain stage during reversal loadings.

VI. Joints Confined by Beams

The behavior of a beam-column joint is influenced by several variables, i.e.,

- Concrete strength.
- Arrangement of joint reinforcement.
- Size and quantity of beam/column reinforcement.
- Bond between concrete and longitudinal bars in both beam and column.
- Finally, axial load in the column.

Type 1 joints: As per ACI 352R-02, item (4.2.1.4), the hoop reinforcement can be omitted when the joints are confined by beams framing into the sides of the column. Such confinement may be assumed when:

- Beams frame into all four sides of the joint and each beam width is at least 3/4 of the column width and does not leave more than 100 mm of the column width uncovered on either side of the beams.
- Beams frame into two opposite sides of a joint, and each beam width is at least three quarters of the column width, leaving no more than 100 mm of the column width on either side of the beam. In this case however, horizontal transverse reinforcement should be provided in the perpendicular direction.
- For discontinuous columns, (both roof and mezzanine floor), it is recommended to use at least two layers of vertical transverse stirrups to improve the confinement of the joint. It is adequate to improve bond of beam top bars and consequently led to stable joint stiffness. In recent ECP-RC-18, the condition of using at least two layers of vertical transverse stirrups for discontinuous columns, is not taken into consideration, despite its importance for improving joint performance during earthquakes.

Confinement Reinforcement

Confinement of the joint core is intended to maintain the integrity of joint concrete, to improve joint concrete toughness, and to reduce the rate of stiffness and strength deterioration (ACI 352R-02). Therefore, the code provisions emphasize the importance of the confinement of joint core through suggesting that the column confinement steel should be continued into the joint. The required area of confinement reinforcement in the joint, when spiral and rectangular hoops reinforcement are used, is as mentioned in equations (6 &7) respectively.

$$\rho_s = 0.12\ \frac{f_c^{'}}{f_{yh}}\,(6a)$$

But should not less than,

$$\rho_s = 0.45\ \left(\frac{A_g}{A_c} - 1\right)\frac{f_c^{'}}{f_{yh}} \qquad (6b)$$

$$A_{sh} = 0.30\ \frac{s_h b_c f_c^{'}}{f_{yh}}\left(\frac{A_g}{A_c} - 1\right) \qquad (7a)$$

But should not less than,

$$A_{sh} = 0.09\ \frac{s_h b_c f_c^{'}}{f_{yh}} \qquad (7b)$$

Type 2 joints: for connections composed of members that are part of the primary system for resisting seismic lateral loads, the following precautions should be considered:

- If the joint is confined by beams, transverse reinforcement equal to at least half the confining reinforcement required at the end of the column should be provided within the depth of the shallowest framing member.
- The spacing of the hoops should not exceed the least of 150 mm, ¼ minimum column's dimension, and six times the diameter of longitudinal column bars, {150mm, b_c/4, 6d_{bar}}.
- The ties within the joint should be provided as closed hoops with the ends bent as 135^o hooks.

In recent ECP-RC-18, the required area of confinement reinforcement in the joint, is stated only when rectangular hoops reinforcement was used. But for spiral hoops reinforcement, there is no any guideline for designers.

Anchorage of Bars at Joints

In interior joints, the flexural reinforcement in beam entering one face of the joint is usually continued through the joint to become the flexural steel for the beam entering the opposite side. However, in exterior or corner joints, beams opposite to face, the flexural reinforcement will not continue beyond the joint, therefore, it is extent and hooked downwards till the mid height of the joint. For both joint types, the minimum development length should not less than the smaller of both 8 d_b and 150mm.

Type 1 joints: the critical section for development of yield strength of the beam bars may be taken at the face of the column. The development length of top beam bars should be computed as in equation (8a).

$$l_{dh} = \frac{f_y d_b}{4.2\sqrt{f_c^{'}}} \qquad (8a)$$

Type 2 joints: During seismic loading, stresses due to moment reversals take place at beam-column connections. These reversals stress causes cracks followed by spalling of concrete column cover. Due this behavior, the ACI 352R-02 suggests that the critical section for development length be taken at the face of the confined column core. It should be noted that, due to formation of plastic hinges in beams, the following are recognized:

- A splitting crack may appear along the top bar of beams as shown in Fig. (8a).
- The bond stress distribution around the bar will not be uniform.
- Hooks should be located within 50 mm of the confined core, and turned downwards and extended towards the mid-depth of the joint, as shown in Fig. (8c). If the beam has more than one layer of flexural reinforcement, the tails of subsequent layers of bars should be located within 3d_b of the adjacent tail.

The development length of top beam bars should be computed as in equation (8b), Where, α is the stress multiplier of longitudinal bars and equals 1.25.

$$l_{dh} = \frac{\alpha\, f_y d_b}{6.20\sqrt{f_c^{'}}} \qquad (8b)$$

In recent [ECP-RC-18], the bar turned downwards, (tail length), within outer surface of hoops, of the confined core by length not less than 12d_{bar}. Added to this, Egyptian code stated one equation for both types, as in equation (9).

$$l_{dh} = 0.2\frac{f_y\ /\gamma_s}{\sqrt{f_{cu}/\gamma_c}}d_b \quad = 0.2\ \sqrt{\frac{\gamma_c}{3/4}}\frac{1}{\gamma_s}\frac{fy}{\sqrt{f_c^{'}}}d_b \qquad (9)$$

In equation (9), the material strength reduction factors γc and γs equals 1.5 and 1.15 respectively, and consequently, the constant equals to (1/4.066). That is, Egyptian code give development length larger than ACI codes by 3.30%, and 22% for types 1 and 2 respectively.

Beam and Column Bars Passing Through the connection

The factors that effects on the bond response of bars passing the beam-column joint, are as follows:

- Using stirrups perpendicular to direction of embedded bar, increase its confinement, and subsequently improve bond performance under seismic conditions.
- It's preferable to use bars diameter d_b less than No. 14, because there is insufficient data to provide guidelines for their behavior under reversal loads.
- Using deformed bars improves resistance against slippage and increases the bond strength.
- Increasing the distance between bars lead to increase the bond strength, due to prevent the interaction of concrete cracks around the bar to adjacent one.

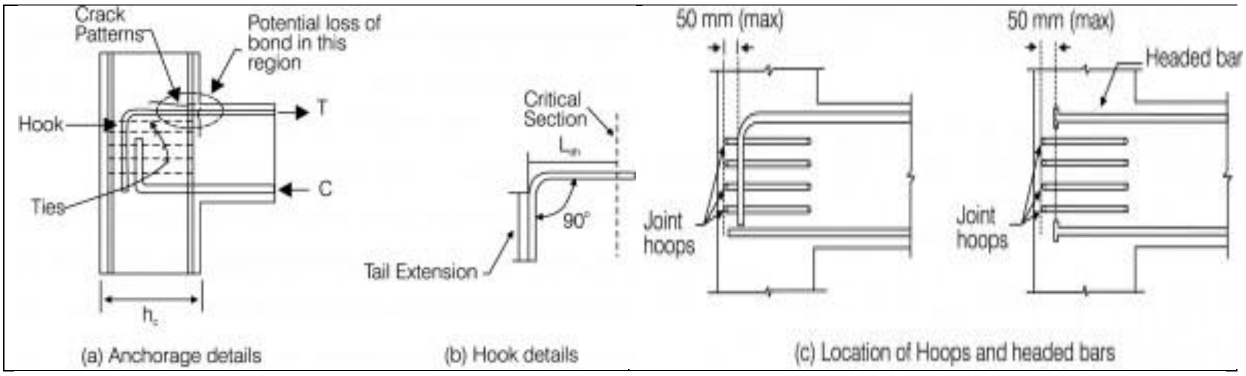


Figure no 8: Development length of longitudinal member reinforcement, [3]

ACI 352R-02 has the following limitation to control slippage of beam and column bars that passing through the connection:

Type 1 Joints:

No recommendations are made.

Type 2 Joints:

case (1): "The column is wider than beams"

$$\frac{h_{column}}{d_{b(beam\ \ bars)}} \geq 20\frac{f_y}{420} \qquad (10a)$$

and

$$\frac{h_{beam}}{d_{b(column\ \ bars)}} \geq 20\frac{f_y}{420} \qquad (10b)$$

Case (2): "Wide beam construction"

$$\frac{h_{column}}{d_{b(beam\ \ bars)}} \geq 24\frac{f_y}{420} \qquad (11)$$

ACI 352R-02 states the following precautions:

- The concrete tensile strength and the specified steel yield stress influence anchorage capacity of longitudinal bars.
- Bar slippage within the joint occurs at 20 d_b length, therefore, the stiffness and energy dissipation capacity reduces at the joint zone. Recent researches recommend using anchorage lengths of 24 to 28 d_b because the joint behavior is better than those of 16 to 20 d_b.
- The [ECP-RC-18] recommends 20 d_b, for controlling slippage of beam and column bars that passing through the joint.

VII. Codes Comparison

Finally, the comparison between ACI352R-02, and ECP-RC-18 codes can be summarized as follows:

The essential requirements in both codes for the satisfactory performance of a joint in an RC structure, during earthquakes, can be stated in the following steps:

a) Member sizes

In seismic conditions involving reversed cyclic loading, anchorage requirements assume great importance in deciding the sizes of the members. Also, the requirement of adequate flexural strength of columns to ensure beam yield mechanism affects the member sizes. The relevant expressions suggested by three codes with regard to development length and flexural strength ratios are summarized in Table 3.

Table no 3: Code requirements that influence the size of the members

Parameter	ECP-RC-18[1]	ACI352R-02[3]
Flexural strength of column	$\sum M_{Pc} \geq 1.20 \sum M_{Pb}$	$\sum M_{Pc} \geq 1.20 \sum M_{Pb}$
Development length (Interior joint)	$h_c/ d_{bar} \geq 20$ [item(6-8-2-3-2-b)]	$h_c/ d_{bar} \geq 24 \sim 28$
Development length (Exterior joint)	Eq. (9) for type 2 Only	Eq.(8a) = Type1 Eq.(8b) = Type2

b) Depth of member for exterior joint

The anchorage and development length of the bars within the joint is usually defined with respect to a critical section located at a distance from the column face where the bars pass through the joint. The critical section refers to the section from where the development length would be considered effective and not affected by yield penetration and deterioration of bond. The expressions for horizontal development length required for exterior joints by the three codes are shown in Table 4.

Table no 4: Code requirements for hook bar in exterior joint

Parameter		ECP-RC-18[1]	ACI352R-02[3]
Critical section from face of column	Type 1	No recommendation	At outside edge of beam
	Type 2	Outside of the joint hoops	50mm of the extent of the confined core
Tail extension		12 d_{bar}	Mid height of the joint

c) Effective joint area

The effective joint area, A_j is the area resisting the shear within the joint and is contributed by the framing members in the considered direction of loading. The depth of the joint, h_j is taken as equal to the depth of the column, h_c. The width of the joint, b_j as per different codes is given in Table 5.

Table no 5: Effective width of joint, b_j ,Figure no 3

Parameter	ECP-RC-18[1]	ACI352R-02[3]
$b_{column} > b_{beam}$	Min. of (b_b+b_c) or (b_b+2x)	Min. of (b_b+b_c) or (b_b+2x)
$b_{column} < b_{beam}$	No recommendation	b_{column}

d) Detailing for shear reinforcement

Detailing features relevant to beam-column joints are concerned with aspects such as spacing of longitudinal and transverse reinforcement and development length for embedded bars. The spacing requirements imposed by the three codes are summarized in Table 6.

Table no 6: Spacing requirements for horizontal and vertical transverse reinforcements, mm

Parameter	ECP-RC-18[1]	ACI352R-02[3]
Spacing between horizontal hoops	{150mm, $b_c/2$, 24 d_{hoop} , 8 d_{cbar}}	{150mm, $b_c/4$, 6 d_{cbar}}
Spacing for vertical U-shaped stirrups, at discontinuous end of a column	No recommendation	- At least two layers - {<300mm, Type 1} - {<150mm, Type 2}

VIII. Conclusions

The performance of framed structures depends not only upon the individual structural elements but also upon the integrity of the joints. However, despite the significance of the joints in sustaining large deformations and forces during earthquakes, ECP-RC-18 [1]guidelines are not full modified for their design and detailing until recently. Only some provisions have been included based on ACI 318-14 [2] and ACI 352R-02 [3] provisions, as obvious from Tables (3 to 6).

From all aboe, it could be concluded that:

- The factor of confinement coefficient "k_j" (in for cases (A1&2-type 2 and B1&2 type 1) is overestimated by 7%. than values estimated in ACI codes.
- The [ECP-RC-18] stated that the length of column dimension, in the direction of the beam that framed into the joint, is not less than 20 d_{bar}, item (6-8-2-3-2-b). Despite, the required length for controlling slippage of beam and column bars that passing through the joint is found between 24 to 28 d_{bar}, i.e., increase the length by 20 to 40%, as stated in ACI352R-02, item (4.5.5).
- From equation (9), ECP-RC-18 code gives development length larger than ACI codes by 3.30%, and 22% for types 1 and 2 respectively.

- In recent ECP-RC-18, the required area of confinement reinforcement in the joint, is stated only when rectangular hoops reinforcement was used. But for spiral hoops reinforcement, there is no any guideline for designers.
- It should be noted that, the condition of using at least two layers of vertical transverse stirrups for discontinuous columns, is not taken into consideration in recent ECP-RC-18, despite its importance for improving joint performance during earthquakes.
- The ECP-RC-18, does not state any guidelines for designers to deal with eccentric beam framed into the joint.
- The codes place high importance for design and detailing of joints especially in seismic zones through providing adequate anchorage of longitudinal bars and confinement of core concrete in resisting shear.
- The depth of column in interior joint is required to be larger as compared to that in exterior joint from anchorage point of view.
- The detailing requirements ensure adequate confinement of core concrete and preclude the buckling of longitudinal bars. The horizontal transverse reinforcements are to be distributed within the joint to resist the diagonal shear cracking and to contain the transverse tensile strain in core concrete.
- It is advised that, the joints in buildings that located in earthquake zones and were built before the development of current design guidelines should be redesigned and strengthened if needed.

References

[1]. Egyptian Code of Practice for Design and Construction of Concrete Structures, ECP-RC, (203-2018), by Ministry of Housing and Development, Housing and Building National Research Centre, Cairo, EGYPT.

[2]. ACI318-14. "Building code requirements for structural concrete (ACI 318M-14) and commentary (ACI 318RM-14)". ACI standard, American concrete Institute, Farmington Hills, (2014).

[3]. ACI352R-02. "Recommendations for Design of Beam-Column Connections in Monolithic Reinforced Concrete Structures", ACI-ASCE Committee 352, (2002).

[4]. Y. C. Choi, et al. "Development of a shear strength equation for beam-column connections in reinforced concrete and steel composite system". International Journal of Concrete Structures and Materials, Vol.11: pp.185-197, (2017).

[5]. Uma, S.R. and S. K. Jain, "Seismic Design of Beam Column Joints in RC Moment Resisting Frames- Review of Codes", Structural Engineering and Mechanics, Vol. 23, No.5, pp. 579-597, (2006).

[6]. Subramanian, "Design of Reinforced Concrete Structures", Oxford University Press, New Delhi, (2015).

[7]. Paulay, and M. Priestley. "Seismic design of reinforced concrete and masonry buildings". John Wiley and Sons, New York, (1992).

[8]. Rai, D.C. and AlpaSheth, "Seismic Design of Beam Column Joints", The Indian Concrete Journal, Vol. 76, No.6, , pp.376-378 June (2002).

[9]. Uma, S.R. and A. M., Prasad, "Seismic Behavior of Beam Column Joints in Reinforced Concrete Moment Resisting Frames", Document No. IITK-GSDMA-EQ31-V1.0, 29 pp, (2005) .

[10]. Hanson, N.W., and H.W. Connor (1967), 'Seismic Resistance of Reinforced Concrete Beam-Column Joints", Journal of Structural Div., ASCE, Vol. 93, No. ST5, May 1967, pp. 533-559.

[11]. Moehle, J.P., J.D. Hooper, and C.D. Lubke, "Seismic Design of Reinforced concrete Special Moment Frames: A Guide for Practicing Engineers", NEHRP Seismic Design Technical Brief No.1, NIST GCR 8-917-1, Gaithersburg, MD, USA, 27 pp, (2008).

[12]. Kim, J., and J.M. LaFave, (2008) 'Probabilistic Joint Shear Strength Models for design of RC Beam-Column Connections, ACI Structural Journal, Vol. 105, No. 6, Nov.-Dec. 2008, pp. 770- 780.

[13]. Hwang, S-J., H-J. Lee, T.-F. Liao, K.-C. Wang, and H.-H. Tsai, (2005), " Role of hoops on Shear strength of Reinforced Concrete Beam-Column Joints", ACI Structural Journal, Vol.102, No.3, pp.445-453, May-June (2005).

[14]. S. K. Ghosh. "Significant changes from the 2011 to the 2014 edition of ACI 318". Precast/Pre-stressed Concrete Institute, c/o PCI Journal, 200 W. Adams St., Suite 2100, Chicago, IL60606:56-79, (2016).

[15]. EN 1998-1:2004:E, "Eurocode 8: Design of structures for earthquake resistance – Part 1: General rules, seismic actions and rules for buildings", European Committee for standardization, Brussels, Incorporating corrigendum July (2009).

Khaled Z. Sayed. "Design Requirements of Reinforced Concrete Beam-column Joints in International Codes and ECP-RC-2018." IOSR Journal of Mechanical and Civil Engineering (IOSR-JMCE), vol. 16, no.6, 2019, pp. 16-27