Document No. :: IITK-GSDMA-EQ32-V1.0
Final Report :: A - Earthquake Codes
IITK-GSDMA Project on Building Codes

Seismic Behavior of Beam Column Joints in Reinforced Concrete Moment Resisting Frames

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SEISMIC DESIGN OF BEAM-COLUMN JOINTS IN RC MOMENT RESISTING FRAMES – REVIEW OF CODES

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Abstract

The behaviour of reinforced concrete moment resisting frame structures in recent earthquakes all over the world has highlighted the consequences of poor performance of beam column joints. Large amount of research carried out to understand the complex mechanisms and safe behaviour of beam column joints has gone into code recommendations. This paper presents critical review of recommendations of well established codes regarding design and detailing aspects of beam column joints. The codes of practice considered are ACI 318M-02, NZS 3101: Part 1:1995 and the Eurocode 8 of EN 1998-1:2003. All three codes aim to satisfy the bond and shear requirements within the joint. It is observed that ACI 318M-02 requires smaller column depth as compared to the other two codes based on the anchorage conditions. NZS 3101:1995 and EN 1998-1:2003 consider the shear stress level to obtain the required stirrup reinforcement whereas ACI 318M-02 provides stirrup reinforcement to retain the axial load capacity of column by confinement. Significant factors influencing the design of beam-column joints are identified and the effect of their variations on design parameters is compared. The variation in the requirements of shear reinforcement is substantial among the three codes.

Submitted to a Journal for possible publication

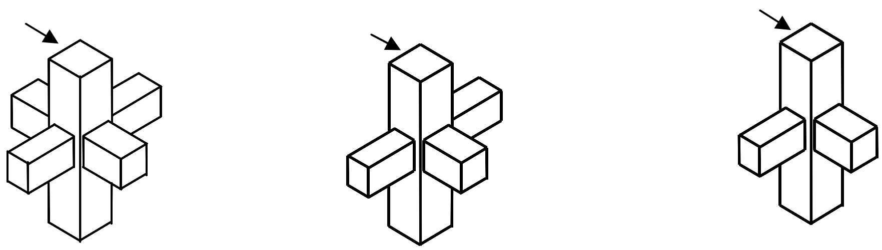
1. Introduction

Beam column joints in a reinforced concrete moment resisting frame are crucial zones for transfer of loads effectively between the connecting elements (i.e. beams and columns) in the structure. In normal design practice for gravity loads, the design check for joints is not critical and hence not warranted. But, the failure of reinforced concrete frames during many earthquakes has demonstrated heavy distress due to shear in the joints that culminated in the collapse of the structure. Detailed studies of joints for buildings in seismic regions have been undertaken only in the past three to four decades. It is worth mentioning that the relevant research outcomes on beam column joints from different countries have led to conflicts in certain aspects of design. Coordinated programmes were conducted by researchers from various countries to identify these conflicting issues and resolve them (Park and Hopkins, 1989). Nevertheless, it is imperative and informative to bring out the critical aspects with respect to design of seismic joints adopted by various international codes of practice.

This paper presents a comprehensive review of the design and detailing requirements of interior and exterior joints of special moment resisting reinforced concrete frames, with reference to three codes of practices: American Concrete Institute (ACI 318M-02), New Zealand Standards (NZS 3101:1995) and Eurocode 8 (EN 1998-1:2003). The discussions with respect to Eurocode are pertaining to 'High' ductility class defined by that code.

2. JOINTS IN REINFORCED CONCRETE MOMENT RESISTING FRAMES

Beam column joints are generally classified with respect to geometrical configuration and identified as interior, exterior and corner joints as shown in Fig.1. This paper focuses to bring out the fundamental differences in the mechanisms of beam longitudinal bar anchorages and the shear requirements, two types of joints such as interior joint and exterior joint are considered. With respect to the plane of loading, an interior beam-column joint consists of two beams on either side of the column and an exterior beam-column joint has a beam terminating on one face of the column.



(a) Interior Joint (b) Exterior Joint (c) Corner Joint

Fig.1. Types of Joints in a Moment Resisting Frame

3. DESIGN APPROACH BY CODES

In reinforced concrete moment resisting frame structures, the functional requirement of a joint, which is the zone of intersection of beams and columns, is to enable the adjoining members to develop and sustain their ultimate capacity. The demand on this finite size element is always severe and more complex due to the possible two-way actions in three-dimensional frame structures. However, the codes consider one direction of loading at a time and arrive at the design parameters for the joint.

3.1 General Criteria

The basic requirement of design is that the joint must be stronger than the adjoining hinging members, usually the beams or columns. It is important to see that the joint size is adequate early in the design phase, since otherwise the column or beam size may need to be changed to satisfy the joint strength or anchorage requirements. The design of beam column joints is predominantly focused on providing shear strength and adequate anchorage within the joint. The mechanisms involved in resisting the bond and shear for interior and exterior joints have been discussed in detail by Paulay and Priestley, 1992.

Judicious detailing of reinforcement is of paramount importance to obtain a ductile response of reinforced concrete structures during a severe earthquake. One of the objectives of detailing is to ensure that the full strength of reinforcing bars, serving either as principal flexural or transverse reinforcement, can be developed under the most adverse condition that an earthquake may impose. Detailing features relevant to beam-column joints are concerned with aspects such as transverse reinforcement for shear strength and confinement, spacing of column longitudinal reinforcement and development length for embedded bars.

In a global sense, the design procedure of beam-column joints consists of the following steps:

- Arrive at the preliminary size for members based on anchorage requirements for the chosen longitudinal bars.
- Ensure adequate flexural strength of columns to get the desired beam yielding mechanism.
- Arrive at the design shear force for the joint by evaluating the flexural overstrength of the adjacent beams and corresponding internal forces. The simultaneous forces in the column that maintain joint equilibrium must also be determined. From these, the joint shear force demand can be calculated.
- Obtain effective joint shear area from the adjoining member dimensions.
- Ensure that the induced shear stress is less than the allowable stress limit. The allowable shear stress limit is expressed as a function of the compressive strength or diagonal tensile strength of concrete. If not satisfied, alter the associated member dimensions, viz., width of the beam or depth of the column.
- Provide transverse reinforcements both as confining reinforcement and as shear reinforcement.
- Provide sufficient anchorage for the reinforcement passing through or terminating in the joint.

The above listed points are elaborated in sequence and discussed in detail with respect to code provisions. In this paper, the variables involved in the code provisions are expressed with the help of commonly adopted notations and symbols, for convenience, and may be different from the specific notations used by the three codes.

3.2 Member Sizes

In seismic conditions involving reversed cyclic loading, anchorage requirements assume great importance in deciding the sizes of the members. This is because the limiting bond stress around the longitudinal bar is to be satisfied by the development length available within the member.

3.2.1 Depth of Member for Interior Joint

In an interior joint, the force in a bar passing continuously through the joint changes from compression to tension. This causes push-pull effect with distribution of bond stress as shown in Fig. 2. The severe demand on bond strength necessitates that adequate development length for the bar be made available within the depth of the member. In other words, for the longitudinal bar of the beam the development length should be provided by the column depth and vice versa. In recognition of this, the codes limit the ratio between the bar diameter and the member depth. By adopting smaller diameter bars which require reduced development length, the sizes of the members can be controlled.

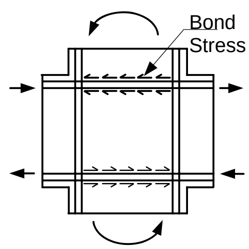


Fig. 2 Bond Condition in an Interior Joint

ACI 318M-02 suggests that where longitudinal beam reinforcement extends through an interior joint, the column depth, h_c shall not be less than 20 times the diameter of the largest longitudinal bar.

$$h_c/d_b \geq 20 \tag{1}$$

where d_b is the diameter of the longitudinal beam bar to be anchored and h_c is the width of the column parallel to the beam

The purpose of the recommended value for h_c/d_b is to provide reasonable control on the amount of potential slip of the beam longitudinal bars through the joint. However, bar slippage may occur even with column depth of 20 d_b. Slippage of bars considerably reduces the stiffness and energy dissipation capacity of the connection region. Therefore, longer development lengths are desirable, particularly when combined with high shear stresses and low values of ratios of column flexural strength to beam flexural strength (Leon, 1990).

NZS 3101:1995 gives the following expression relating the bar diameter and the member depth. The expression explicitly involves the parameters that affect the bond performance such as axial load, condition of concreting done near the bar and material strengths. The code suggests an expression in the form of bar diameter to column depth ratio as

$$\frac{d_b}{h_c} \leq 6\left[\frac{\alpha_t\,\alpha_p}{\alpha_s}\right]\alpha_f\,\frac{\sqrt{f_c'}}{\alpha_0\,f_y} \tag{2}$$

where f_c' is characteristic cylinder strength of concrete; f_y is characteristic yield strength of steel, α_f is 1.0, for one-way frame loading and 0.85 for two-way frame loading; α_0 is 1.25 for overstrength for yield strength of steel at strain hardening; α_t is 0.85 for a top beam bar where more than 300 mm of fresh concrete is cast below the bar and 1.0 for all other cases and α_p is a factor to include beneficial effect of compression on column where,

$$\alpha_p = \frac{N^*}{2\,f_c'\,A_g} + 0.95; \qquad 1.0 \leq \alpha_p \leq 1.25 \tag{3}$$

N* is the minimum axial compression load on the column consistent with the governing ultimate limit state of load combination. When the area of bottom bars, A_s', is less than that of the top bars at a section, the bottom bars can be subjected to higher stresses in compression for the applied moment at the joint face. Therefore these bars require a higher anchorage length to transmit the bond force. The parameter α_s accounts for this as follows:

$$\alpha_s = 2.55 - \frac{A_s'}{A_s}\,; \tag{4}$$

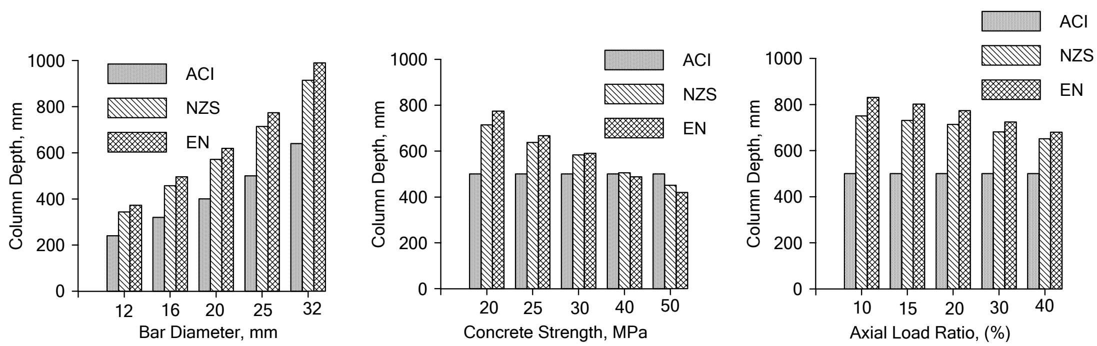
where $0.75 \leq \frac{A_s'}{A_s} \leq 1.0$. For beam bars which are part of larger area A_s, α_s is 1.55 taking A_S'/A_S as unity.

EN 1998-1: 2003 recommends an expression similar to that in the NZS code by considering the effect of axial load, material strength and ratio of compression to tension reinforcement. Anchorage of longitudinal bars for interior beam column joints high ductility class (DCH) must satisfy the following expression:

$$\frac{d_b}{h_c} \le \frac{7.5\, f_{ctm}}{\gamma_{Rd}\; f_{yd}} \frac{1+0.8\nu_d}{1+0.75\, k_D\, \rho'/\rho_{\max}} \tag{5}$$

where f_{ctm} is the mean value of tensile strength of concrete given as $0.3 f_c'^{\,(0.667)}$; f_{yd} is design value of yield strength of steel; $\rho'/\rho_{\max}$ is ratio of compression reinforcement to maximum tension reinforcement of the beam framing in the joint; k_D is 1 for high ductility class structure; γ_{Rd} is model uncertainty factor for the design value of resistances, taken as equal to 1.2 due to overstrength at strain hardening of longitudinal steel in the beam; and f_{cd} is design value of cylinder strength of concrete and ν_d is normalised design axial force in column expressed as $\nu_d = \frac{N_{Ed}}{f_{cd}\; A_g}$ where N_{Ed} taken with its minimum value for seismic load combination.

Three important parameters influencing the column depth are bar diameter d_b, concrete compressive strength, f_c' and normalized axial force ratio on column, ν_d. The effect of each parameter on column depth has been compared by keeping the other two as constant and is shown in Fig.3. For illustrative purpose, values for concrete cylinder strength as 20 MPa, characteristic yield strength of steel as 415 MPa, an axial load ratio of 0.2 and $\rho'/\rho_{\max}$ as 0.5 are assumed to study the variation of column depth for different bar diameters. Fig. 3 (a) shows variations in required column depths with diameter of beam bars. EN 1998-1:2003 consistently shows larger column depth. The ratio of column depth to diameter of the bar is 20 as per ACI 318M-02 code, above 29 as per NZS 3101:1995 code recommendation and more than 31 for EN 1998-1:2003 specifications. Fig. 3 (b) shows the variation in required column depth with concrete grade as per the three codes for a longitudinal bar of size 25mm diameter. ACI 318M-02 does not consider the concrete strength and hence the column depth required remains constant. NZS 3101:1995 reduces the depth of column moderately with increase in concrete grade whereas the rate of reduction is higher as per EN 1998-1:2003.



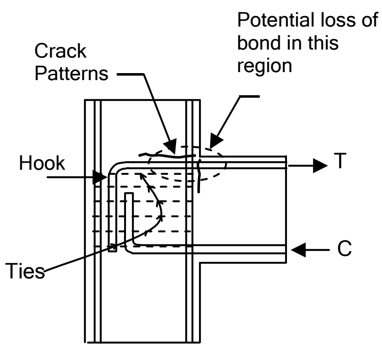
(a) Effect of Bar Diameter (b) Effect of Concrete Strength (c) Effect of Axial Load Ratio

Fig.3 Variation of Column Depth for Interior Joint

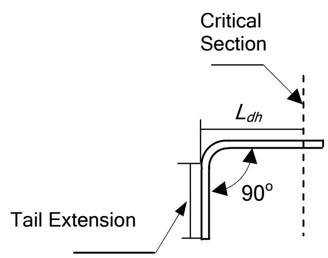
The axial compression load on column improves the confinement of joint core to some extent which in turn improves the bond condition within joint core (Paulay and Priestley,1992). This effect is included in the determination of column depth by NZS and EN codes and the comparison is shown in Fig. 3(c). NZS provision accounts for a reduction of 4.5% and EN expression reduces up to 6% for 10% increase in column axial load ratio. The above comparisons indicate that the ACI provision of $h_c/d_b \geq 20$ is less conservative as compared to NZS and EN codes except when the grade of concrete is considerably higher.

3.2.2 Depth of Member for Exterior Joint

In exterior joints the beam longitudinal reinforcement that frames into the column terminates within the joint core. Fig. 4 shows the typical anchoring of beam bars and the bond deterioration in an 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 enter into 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. All the three codes define critical sections differently depending on factors such as the ductility level sought and extent of yield penetration. ACI 318M-02 defines the anchorage length from the face of the joint. NZS suggests the critical section to be located at minimum of half the column depth or 8 db from the column face and EN code suggests a distance of 5 d_b from column face for high ductility class of structures. The anchorage for the longitudinal bars terminating into the joint core is usually provided in the form of ACI standard 90^o hooks, which include a horizontal development length, L_{dh}, and a tail extension as shown in Fig. 4 (b). ACI 318M-02 suggests the length of tail as 12 d_b, NZS 3101:1995 and EN 1998-1:2003 require a minimum length of 12 d_b and 10 d_b respectively.



(a) Anchorage Details



(b) Hook Details

Fig.4 Details of Exterior Joint

The ACI 318M-02 provides expression for horizontal development length L_{dh} as

$$L_{dh} = \frac{f_y\, d_b}{5.4\sqrt{f_c'}} \tag{6}$$

NZS 3101:1995 gives the expression of horizontal development length as

$$L_{dh} = 0.24\ \alpha_b\ \alpha_1\ \alpha_2\ \frac{f_y\ d_b}{\sqrt{f_c'}} \tag{7}$$

where α_b is reduction factor equal to the ratio of required flexural reinforcement to provided flexural reinforcement which is considered as 1.0 for the members subjected to earthquake forces; α_1 is 0.7 for 32 mm bars or smaller with side cover normal to the plane of the hooked bar not less than 60 mm and cover on the tail extension of 90^o hooks not less than 40 mm and 1.0 in all other cases; α_2 is 0.85 for confinement by closed stirrups or hoops spacing of 6 d_b or less and 1.0 in all other cases.

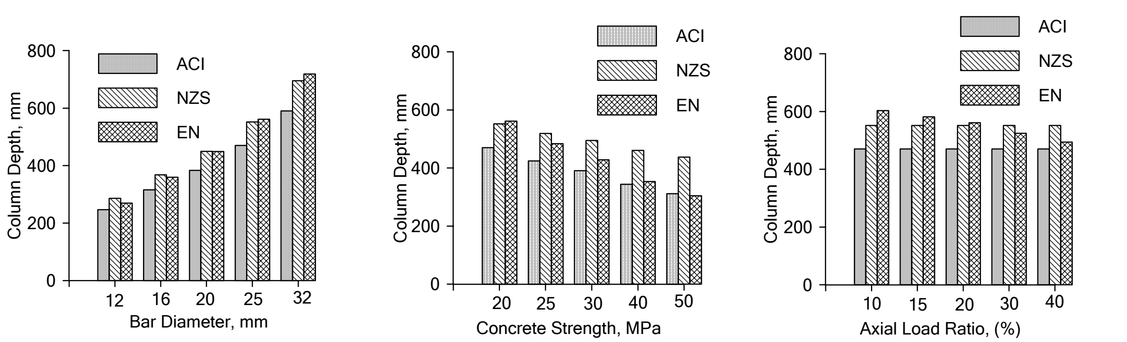
Both ACI and NZS codes specify the minimum development length to be not less than the smaller of 8 d_b or 150mm.

EN 1998-1:2003 expression for anchorage requirements in the case of exterior joint is in the form of beam bar diameter to column depth ratio. It considers the effect of axial load on the column. The following expression gives directly the required depth of column, h_c instead of horizontal development length, L_{dh}.

$$\frac{d_b}{h_c} \leq \frac{7.5\ f_{ctm}}{\gamma_{Rd}\ f_{yd}}\left(1+0.8\nu_d\right) \tag{8}$$

where the parameters are defined in equation (5).

A comparison study has been done for exterior joints to associate the effect of bar diameter, concrete compressive strength and axial load ratio on column depth. For illustrative purpose, the parameters in the suggested expressions of each codes are assumed to have such values as suggested by the respective codes that column depth arrived are the minimum possible. The horizontal development length L_{dh} is measured from the critical section conditions as per each code and the column depth is arrived adding a concrete cover of 40mm.



(a) Effect of Bar Diameter (b) Effect of Concrete Strength (c) Effect of Axial Load Ratio

Fig.5 Variation on Column Depth for Exterior Joint

The deviation in the values of required minimum column depth may also be attributed to the difference in the critical section consideration from the column face. From Fig. 5 (a), it can be seen that for favourable confinement conditions and good cover with material properties as assumed for interior joints, the horizontal development length required as per ACI 318M-02 is 17 d_b and the same required by NZS 3101:1995 is 21 d_b. While it is of more relevance to compare the column depth required for a given bar diameter, it is observed that h_c/d_b will not be a constant ratio and the ratio is slightly high for smaller diameter bars than for larger diameter bars for obvious reasons. For example, as per ACI 318M-02, the column depth required is above 19.5 d_b for 16 mm diameter bars which amounts to 312 mm and around 18 d_b for 32 mm bar which equals to 576 mm. A similar explanation holds good for the values obtained as per NZS 3101:1995 the ratio h_c/d_b varies from 24 d_b for smaller bars to 22 d_b for larger bars. EN 1998-1:2003 gives a constant h_c/d_b ratio equal to 22.4 d_b.

The effect of higher concrete strength in reducing the depth of column is largely reflected in EN 1998-1:2003 provisions than the other two codes as shown in Fig. 5 (b). The effect of axial load is not considered by ACI 318M-02 and NZS 3101:1995 in predicting the horizontal development length and hence column depth remains constant. However, EN 1998-1:2003 includes the effect of axial compression on column and shows a reduction of 6% in column depth for 10% increase in axial load ratio and is shown in Fig. 5 (c).

3.3 Flexural Strength of Columns

The codes recommend expressions to preclude formation of plastic hinges in columns which essentially aim at providing stronger columns with capacity more than the flexural strength of beams obtained considering over strength factors. A rigorous interpretation of expressions requires calculation of the moments at the centre of the joint. These moments correspond to development of the design values of the moments of resistance of the columns or beams at the outside faces of the joint, plus a suitable allowance for moments due to shears at the joint faces. However, the loss in accuracy is minor and the simplification achieved is considerable if the shear allowance is neglected. ACI 318M-02 and EN 1998-1:2003 consider this approximation acceptable whereas NZS 3101:1995 suggests the expression with respect to centre of joint. For connections with beam framing in from two mutually perpendicular directions, this provision should be checked independently in each direction.

ACI 318M-02 recommends that the sum of the nominal flexural strengths of the column section above and below the joint calculated at the joint faces using the factored axial load that results in the minimum column-flexural strength, should not be less than 1.2 times the nominal flexural strength of the beam sections at the joint faces.

$$\sum M_{n,c} \geq 1.2 \sum M_{n,b} \tag{9}$$

In Eq.(11) M_n represents the nominal flexural strength and the subscripts $_c$ and $_b$ represent column and beam, respectively. In T-beam construction, where the slab is in tension under moments at the face of the joint, slab reinforcement within an effective slab width suggested by the code should be assumed to contribute to the flexural strength of the beams, if the slab reinforcement is developed at the joint face for flexure.

As per the capacity design of columns for flexure NZS3101:1995 requires the design flexural strength of the column to be in excess to the flexural overstrength of adjacent beams. With reference to the centre of the joint, this requirement is given as

$$\sum_{c} M_n \geq 1.4 \sum_{b} \phi_o M_n \tag{10}$$

where ϕ_o is overstrength factor for beams, which may be taken as 1.47 from the overstrength of steel as 1.25 and a strength reduction factor of 0.85. The column moments derived for lateral static forces are likely to be magnified by the participation of higher modes; and this effect is represented by dynamic moment magnification factor, the value of which ranges from 1.3 to 1.8 for one-way frames (Paulay and Priestley, 1992). The code includes this effect by adopting a value 1.4 as given in Eq (10). Hence it can be understood that the design flexural strength of columns are expected to be at least 2.06 times higher than the design flexural strength of adjoining beams. This factor is much greater than those suggested by ACI 318M-02 and EN 1998-1:2003.

EN1998-1:2003 suggests the following condition to be satisfied at all joints:

$$\sum M_{Rc} \geq 1.3 \sum M_{Rb} \tag{11}$$

where $\sum M_{Rc}$ is the sum of the design values of the minimum moments of resistance of the columns within the range of column axial forces produced by the seismic design situation and $\sum M_{Rb}$ is the sum of the design values of the moment of resistance of the beams framing into the joint.

3.4 Shear Force acting on the Joint

The shear forces in the joint is idealized to be acting in horizontal and vertical directions and are basically arrived from the internal forces associated with hinging conditions in the adjoining members as shown in Fig. 6. The equilibrium of forces in horizontal and vertical directions considered independently gives respective horizontal shear force, V_{jh} and vertical shear force, V_{jv} acting in the joint.

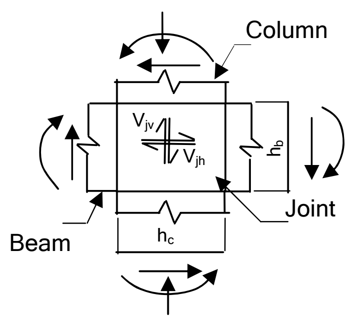


Fig. 6 Shear Forces Acting in the Joint

With the assumption that the beams are designed for plastic hinge formations, the flexural overstrength of beams on either side of the joint is evaluated corresponding to positive and negative moment capacities. The contribution of floor slab is considered in the form of effective flange width for the beam. The flexural strength in positive and negative bending is arrived at and is referred to as M_{1o} and M_{2o}. The column shear can be calculated as

$$V_{col} = \frac{M_{1o} + M_{2o}}{(l'_c + l_c)/2} \tag{12}$$

where l'_c and l_c are the heights of the columns above and below the joint.

The shear force demand, V_{jh}, in the horizontal direction can be obtained as the net force acting on a horizontal plane across the joint so as to include the forces from the beam and the shear force in the column.

$$V_{jh} = (A_{s1} + A_{s2})\alpha_o f_y - V_{col} \tag{13}$$

where A_{s1} is top reinforcement in the beam including reinforcement in effective flange width and A_{s2} is bottom reinforcement in the beam.

Similarly, consideration of equilibrium of vertical forces at the joint would lead to expressions for the vertical joint shear force, V_{jv}. However, because of the multilayered arrangement of the column reinforcement, the derivation of vertical stress resultant is more cumbersome. It can be noted that the horizontal shear stress in the joint should equal the corresponding vertical shear stress and with assumption of uniform distribution of the stresses along each face of the joint the following equation can be written as

$$\frac{V_{jh}}{b_j\, h_c} = \frac{V_{jv}}{b_j\, h_b} \tag{14}$$

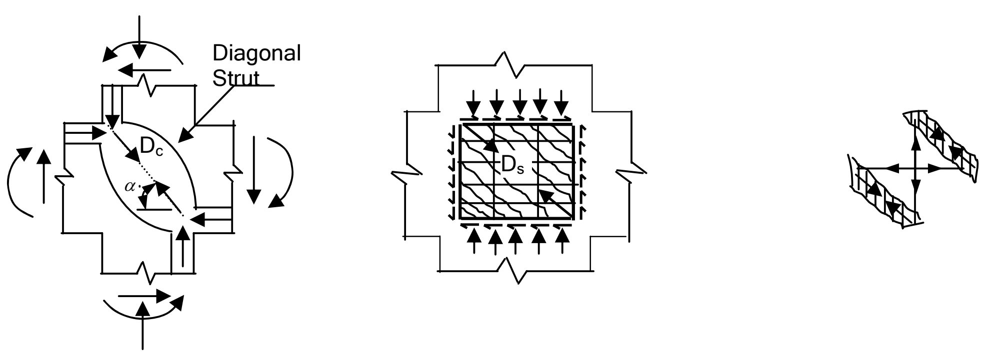
where b_j is effective width of joint (Ref. sec. 3.6); h_b is depth of beam; h_c is depth of column. Hence, for common design situations, it is generally considered sufficiently accurate to estimate vertical joint shear force in proportion to horizontal shear force. This can be expressed as

$$V_{jv} = \left(h_b / h_c\right) V_{jh} \tag{15}$$

3.5 Shear Strength of Joint

The shear forces in the joint region in vertical and horizontal directions develop diagonal compressive and tensile forces within the joint core. The shear transfer mechanisms are very complex since interplay of shear, bond and confinement takes place within the joint. Hence, conflicting views exist between researchers with regard to design parameters of the joint. The model proposed by Paulay, Park and Priestley (1978) considers that the total shear within the joint core is partly carried by a diagonal concrete strut (Fig. 7 (a)) and partly by an idealized truss consisting of horizontal hoops, intermediate column bars and inclined concrete bars between diagonal cracks (Fig. 7 (b),(c)). The strut mechanism is associated with a diagonal force, D_c within the concrete strut developed by major diagonal concrete compression forces formed at the corners of the joint. A substantial portion of the total joint shear (horizontal and vertical) can be resisted by this mechanism. However, the strength of the strut mechanism is reduced by tensile strains perpendicular to the direction of the strut. In such situations confinement of the joint core would help in improving the strength of the strut. The steel forces transferred through bond are introduced into concrete at the four boundaries of the joint core forming a compression field with diagonal cracks in the joint as shown in Fig. 7(b). These forces being in equilibrium generate a total diagonal compression force D_s from all the concrete bars between the diagonal cracks. The mechanism associated is called truss mechanism and is supported by well distributed transverse reinforcement within the joint as shown in Fig. 7(b) and (c).

The diagonal forces D_c and D_s are acting at an angle α with respect to the horizontal axis of the joint. The sum of horizontal components of these forces from both mechanisms gives an estimate of shear resistance in horizontal direction. Similarly, the sum of vertical components gives shear resistance in vertical direction.



(a) Strut Mechanism (b) Truss Mechanism (c) Components of truss

Fig.7 Shear Resisting Mechanisms

When the compression stress within concrete strut is not excessive the strut mechanism is efficient in resisting the shear and the truss mechanism is hardly mobilized. Nevertheless, the transverse reinforcement provides confinement to improve the efficiency of the concrete in the strut mechanism. However, when the core concrete is thoroughly cracked so that no more diagonal tensile stresses can be transferred by concrete, the transverse reinforcements resist shear as shown in Fig. 7 (c). In such situations the contribution of truss mechanism becomes significant, provided good bond conditions are sustained.

In essence, the design of joint to resist the shear force demand is associated with adopting adequate joint dimension to support the strut mechanism and providing adequate transverse reinforcement to take care of truss mechanism. On the other hand, the truss mechanism tends to diminish in case of bond deterioration the transverse reinforcements can no longer be utilized for taking up joint shear. Hence, for design considerations, the compressive strength of the diagonal concrete strut is considered as the reliable source of strength and based on which the codes define nominal shear capacity of the joint. The nominal shear capacity is expressed in terms of allowable stress in concrete and effective joint area, A_j (Ref. 3.6). As the first design step it is verified that the shear force demand in the joint should be less than the nominal shear capacity, if not the dimensions of the joint are to be revised irrespective of the amount of reinforcement available within the joint. Increased joint dimensions improve the strength of strut by increasing the effective joint area and also by reducing the nominal shear stress acting on the joint. Secondly towards the truss mechanism, the joint is provided with necessary shear reinforcement.

3.6 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 width b_j and depth h_j of the joint are arrived at from the member dimensions. 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 1, involving width of beam b_b, width of column b_c, and depth of column h_c. ACI code uses the distance of the column edge beyond the edge of the beam denoted as x, which is considered in the direction of loading. However, it is to be noted that in no case the joint area A_j is greater than the column cross sectional area. NZS and EN codes give identical expressions to determine the width of joint.

Table. 1 Effective width of joint, *bj*

Sl.No	Category	ACI 352R-02	NZS	ENV
1.	$b_c > b_b$	$\min\{b_b + h_c ; b_b + 2x\}$	$\min\{b_c ;\ b_b + 0.5h_c\}$	$\min\{b_c ;\ b_b + 0.5h_c\}$
2.	$b_b > b_c$	b_c	$\min\{b_b ;\ b_c + 0.5h_c\}$	$\min\{b_b ;\ b_c + 0.5h_c\}$

3.7 Nominal Shear Stress of the joint

The level of shear stress, as expressed by nominal shear stress, is an important factor affecting both strength and stiffness of the joint. The codes restrict the nominal shear stress to be less than a fraction of compressive strength of concrete. All three codes evaluate the nominal shear capacity based on strut mechanism and express it as a function of concrete strength irrespective of the amount of shear reinforcement. However, the nominal shear capacity is influenced by the confinement provided by the adjoining members. A beam member that frames into a face is considered to provide confinement to the joint if at least the framing member covers three-quarters of the joint.

ACI 318M-02 sets the nominal shear strength of the joint as a function of only concrete strength, which in turn depends upon the degree of confinement, offered by the members and is given as, $1.7\sqrt{f_c'}\,A_j$ if confined on four faces, $1.25\sqrt{f_c'}\,A_j$ if confined on three faces and $1.0\sqrt{f_c'}\,A_j$ for other cases. Apart from this, the code requires a minimum amount of transverse reinforcement in the joint as shear reinforcement and to provide for confinement of core concrete.

The NZS 3101:1995 has developed recommendations considering contributions from strut and truss mechanisms and has suggested a limiting value of 0.2 f_c', with respect to strut mechanism irrespective of the confinement offered by the framing members.

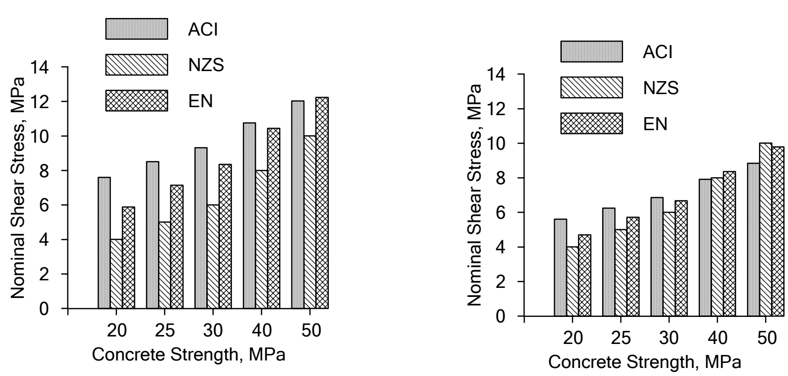
EN 1998-1:2003 also has limited the nominal shear stress, v_{jh} within interior beam column joint to be less than the stress value given by the expression

$$v_{jh} \le \eta f_{cd} \sqrt{1 - \frac{v_d}{\eta}} \tag{16}$$

where $\eta = 0.6\left(1 - \frac{f_c'}{250}\right)$, denotes the reduction factor on concrete compressive strength due to tensile strains in transverse direction.

EN 1998-1:2003 suggests the shear strength of exterior joints to be taken as 80% of the value given by Eq.(16).

The nominal shear stresses calculated on the basis of code recommendations for various grades of concrete are presented in Fig. 8. The interior joint considered in this study is assumed to have beams with sufficient dimension with respect to column dimension to confine the joint. EN 1998-1:2003 includes the axial load ratio, v_d which is assumed as 0.2 in this comparison study. It may be seen in Fig.8 (a) that for interior joints, NZS 3101:1995 consistently yields conservative values. ACI 318M-02 code gives a higher estimate of nominal joint shear capacity compared to the other two codes for interior joints at lower values of concrete strength. For example, for a concrete strength of 20 MPa the nominal shear stress capacity as per ACI 318M-02 is 29% higher than that provided by EN 1998-1:2003 and 90% higher than that suggested by NZS 3101:1995 At higher values of concrete strength, ACI 318M-02 and EN 1998-1:2003 give values 20% higher than that obtained by NZS 3101:1995.



(a) Interior Joint (b) Exterior Joint

Fig. 8 Effect of Concrete Strength on Nominal Shear Stress

Fig.8 (b) shows the comparison of nominal shear stress in exterior joint obtained by the three codes. The exterior joint considered in this study is assumed to have confining beam members on three vertical faces of the column. ACI 318M-02 value is 19% higher than that of EN 1998-1:2003 and 40% higher with respect to NZS 3101:1995 for concrete strength of 20 MPa. At higher strength of concrete ACI 318M-02 is conservative compared to the other two codes.

3.8 Design of Shear Reinforcement

Shear reinforcements in horizontal and vertical directions are designed so as to support the truss mechanism. Usually, the horizontal shear is supported by stirrups and hoops placed in the horizontal direction where as the vertical shear is taken care adequately by intermediate column bars. Since the intermediate column bars are expected to be in compression, the bars will have adequate strength to take tensile stresses developed during shear resisting mechanism.

The horizontal shear supported by the truss mechanism is resisted by transverse steel in the joint. Codes suggest expressions for design of shear reinforcement based on the assumption that plastic hinges develop only in the beams. A minimum amount of transverse reinforcement to support the truss mechanism, equivalent to 40% of the total shear demand, has been suggested by NZS 3101:1995. The provision of joint reinforcement to cater to more than 70% of the total shear does not prevent the compression failure of concrete in the diagonal strut (Ichinose, 1991). However, NZS 3101:1995 sets the upper limit to the shear reinforcement requirement in such a way that the nominal shear stress is less than $0.2 f_c'$, which corresponds to the strength limit of the diagonal strut.

3.8.1 Horizontal Shear Reinforcement

The transverse reinforcement in the joint contributes to confining the core and in resisting shear. ACI 318M-02 does not require an explicit design for shear reinforcement instead the recommended reinforcement is to confine the joint, when the axial load capacity of the column is maintained within the core, after the shell becomes ineffective. The provided confinement is expected to be sufficient for necessary force transfer within the joint.

In members with circular cross section, when spiral reinforcement or circular hoop is used, the volumetric ratio ρ_s, should not be less than

$$\rho_s = 0.12 \frac{f'_c}{f_{yh}} \tag{17}$$

neither should it be less than

$$\rho_s = 0.45 \left(\frac{A_g}{A_c} - 1 \right) \frac{f'_c}{f_{yh}} \tag{18}$$

where f_{yh} is the specified yield strength of the spiral reinforcement but not greater than 420 MPa; A_g is gross section area of section; A_c is area of core of spirally reinforced circular compression member measured to outside diameter of spiral.

In rectangular sections, rectangular hoops and crossties are used as horizontal transverse reinforcement. The efficiency of confinement provided by these hoops is considered to be 0.75 times that provided by the circular hoops. Thus, the total cross-sectional area of stirrups A_{sh}, in each direction of a single hoop, over lapping hoops, or hoops with crossties of the same size in a layer should be at least equal to

$$A_{sh} = 0.3 \left(s\, h''_c \, f'_c / f_{yh} \right) \left[\left(A_g / A_{ch} \right) - 1 \right] \tag{19}$$

but should not be less than

$$A_{sh} = 0.09 \frac{s\, h''_c \, f'_c}{f_{yh}} \tag{20}$$

where s is spacing of transverse reinforcement within the joint; h''_c is cross-sectional dimension of column core measured centre-to-centre of transverse reinforcement; A_{ch} represents cross-sectional area of rectangular structural member measured out-to-out of transverse reinforcement.

NZS 3101:1995 gives expressions for the total amount of shear reinforcement, A_{jh} to be distributed within the joint. In general, the code requires that minimum shear reinforcement is designed for at least 40% of the total horizontal shear force, V_{jh}. However, the design provisions for shear reinforcement as per Eq. (21) and Eq. (23) ensure a lower limit to support the truss mechanism and an upper limit to prevent compression failure of diagonal concrete strut. These are achieved by restricting $6v_{jh} / f_c'$ not to be less than 0.85 and not to be more than 1.2. In other words, that minimum requirement corresponds to a nominal shear stress not less than $0.14 f_c'$ and the maximum requirement corresponds to nominal shear stress not more than $0.2 f_c'$.

The design shear reinforcement for interior joints is arrived at considering the fact that the additional forces from the slab reinforcement, A_{sf} within the effective flange width are transferred as compressive force to the core concrete (NZS 3101:1995) and the strut mechanism would be responsible for taking up this additional force. Hence, the shear reinforcement with regard to truss mechanism is to be provided only for the forces from the top reinforcement A_s* (i.e. A_{s1}-A_{sf}), provided within the web portion of the beam. The area of the total joint shear reinforcement, Ajh corresponding to a nominal shear stress, vjh,, in the interior joint is given by

$$A_{jh} = \frac{6v_{jh}}{f_c'} \alpha_j \frac{f_y}{f_{yh}} A_s^* \tag{21}$$

where $\alpha_j = 1.4$ (or) $\alpha_j = 1.4 - 1.6 \dfrac{C_j N^*}{f_c' A_g}$ (22)

In Eq. (21) α_j takes the value 1.4 in the case of no axial loads on the column. The second expression in Eq. (22) accounts for the effects of the axial compression load acting on the column thereby reducing the amount of shear reinforcement required. The parameter C_j is given as $C_j = \dfrac{V_{jh}}{V_{jx} + V_{jz}}$ where V_{jx} and V_{jz} are total nominal shear force in x and z directions respectively.

In exterior joints, the force from slab reinforcements are not transferred to the core concrete and it is to be resisted by truss mechanism only. Hence, it is necessary to consider the total reinforcement A_{s1} at the top, inclusive of that from the effective flange width in the calculation of design shear reinforcement which is given by

$$A_{jh} = \frac{6\, v_{jh}}{f_c'} \beta \left(0.7 - \frac{C_j\, N^*}{f_c'\, A_g} \right) \frac{f_y}{f_{yh}} A_{s1} \tag{23}$$

where N^* is taken negative with axial tension in which case C_j = 1 must be assumed for one way loaded frames. β is the ratio of the compression beam reinforcement to that of the tension beam reinforcement at an exterior beam column joint, and is not to be taken larger than unity.

EN 1998-1:2003 gives expressions for adequate confinement to be provided to limit the maximum diagonal tensile stress in the core concrete to design value of tensile strength of concrete. The minimum amount of reinforcement required for adequate confinement and to limit diagonal tensile strength of concrete is given as

$$\frac{A_{jh}\, f_{yhd}}{b_j\, h_{jw}} \geq \frac{\left(\dfrac{V_{jh}}{b_j\, h_{jc}} \right)^2}{f_{ctd} + \nu_d\, f_{cd}} - f_{ctd} \tag{24}$$

where V_{jh} is horizontal shear force demand; h_{jw} is distance between top and bottom bars of the beam; h_{jc} is distance between extreme corner bars of the column; f_{ctd} is design value of tensile strength of concrete; f_{yhd} is design value of yield strength of transverse reinforcement;

However, EN code also imposes a requirement to maintain the integrity of the joint after diagonal cracking and hence the necessary reinforcement to be provide for interior is given as

$$A_{jh}\, f_{yhd} \geq \gamma_{Rd} \left(A_{s1} + A_{s2} \right) f_{yd}\, (1 - 0.8 \nu_d) \tag{25}$$

and that for exterior joints is given as

$$A_{jh}\, f_{yhd} \geq \gamma_{Rd}\, A_{s2}\, f_{yd}\, (1 - 0.8 \nu_d) \tag{26}$$

The parameters in Eq. (25) and Eq. (26) are defined in earlier sections. The term A_{jh} in Eq. (24) to Eq. (26) represents the total area of horizontal hoops to be provided within the joint.

An example problem is used here to illustrate the requirements of design shear reinforcement as per the three codes. Details of the members of the joints considered for this study are given in

Table 2. The interior joint has beams with given details on either side of the joint while the exterior joint has one beam terminated in the joint.

Table 2. Section details of interior and exterior joint

	Column	Beam	Slab
Section	625 mm x 625mm	500 mmx 625mm	150 mm thick
Longitudinal Reinforcement	12 – 25 mm dia	Top Reinf: 6- 20mm dia Bot Reinf: 3- 20 mm dia	Top: 10 mm dia at 150 mm spacing Bot: 10 mm dia at 200 mm spacing
Height / Span	3500 mm	5000mm	-

The dimensions of beam and column satisfy the anchorage requirements and the joint has adequate nominal shear capacity. Steel yield strength was taken as 415MPa and horizontal shear reinforcement required by different codes was computed for different grades of concrete. The reinforcements within the effective slab width suggested by ACI 318M-02 has been assumed to be anchored well within the joints and hence accounted for in computing the contribution of negative flexural strength for beams.

3.8.1.1 Design Shear Reinforcement in Interior Joint

The design shear reinforcement A_{jh}, for the entire interior joint as per the three codes are given in Fig. 9.

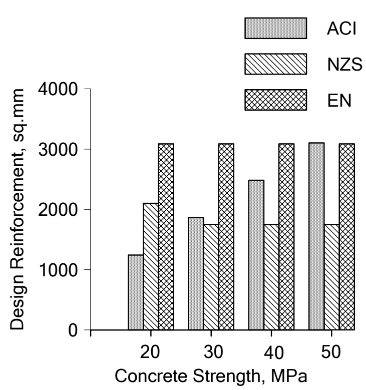


Fig.9 Effect of Concrete Strength on Design Shear Reinforcement in Interior Joint

The expression suggested by ACI 318M-02 as referred in Eq. (19) gives minimum transverse reinforcement required per layer, A_{sh} and this is repeated within the joint adopting the spacing of

100mm to arrive at the required amount for the entire joint, A_{jh}. The amount of reinforcement is increasing proportional to the grade of concrete. To compare the values suggested by EN 1998-1:2003 Eq. (24) and Eq. (25) have been used assuming a constant axial load ratio of 0.2. The reinforcement as per Eq. (24) was not governing in this example problem. Hence the reinforcement requirement as per Eq. (25) has been taken up for comparison with other two codes. The expression does not vary with concrete strength as the expression does not involve any terms related to concrete strength except in the parameter representing axial load ratio, which is assumed as constant. So, the design shear reinforcement is constant for the joint which is only influenced by the forces developed in reinforcements provided in the adjoining beam members. A perusal of the expression given by NZS 3101:1995 indicates that the design shear reinforcement is inversely proportional to the strength of concrete. This leads to an interpretation that higher concrete strength would lower the required amount of shear reinforcement within the joint. However, the calculation of design shear reinforcement is more affected by the limits on the nominal shear stress in concrete to be satisfied and/or on the condition based on minimum shear force to be resisted by the transverse reinforcement as explained earlier. In the chosen example, the design shear reinforcement required for and above concrete strength 30MPa is governed by the constraint imposed by nominal shear stress ratio $\frac{6v_{jh}}{f_c'} \geq 0.85$ and amounts to 1750mm^2 where as 40% of design shear force requires only 1295mm^2. This transverse reinforcement would contribute towards confinement for core concrete to sufficiently resist the shear by strut mechanism.

It may be observed from the above discussion, ACI 318M-02 adopts increasing shear reinforcement with the increase in concrete strength and EN 1998-1:2003 suggests design shear reinforcement based on the forces on the main reinforcement of the beam members which appears to be more stringent even at the lower strength of concrete. The design shear reinforcement values obtained from NZS 3101:1995 clearly indicate even if strength of concrete is increasing, there is a limit for the shear resisting capacity offered by the strut and truss mechanisms and gives appropriate required amount of design shear reinforcement in the joint.

3.8.1.2 Design Shear Reinforcement in Exterior Joint

A comparison on the requirement of design shear reinforcement for exterior joint is done between the three code provisions. ACI 318M-02 recommends Eq. (19) and Eq. (20) both for interior and exterior joints to compute the transverse reinforcement and the design are independent of the amount of shear demand. However, NZS 3101:1995 and EN 1998-1:2003 give explicit provisions for shear reinforcement in exterior joints considering the amount of shear demand within the joint.

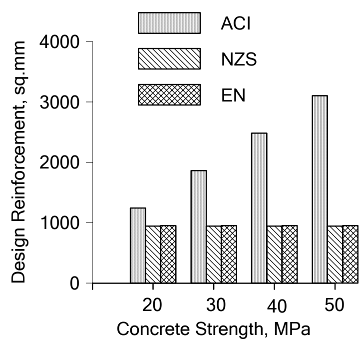


Fig.10 Effect of Concrete Strength on Design Shear Reinforcement in Exterior Joint

The design shear reinforcements computed as per the provisions of all three codes are given in Fig.10. ACI 318M-02 gives constant increase in the design reinforcement with increase in concrete strength. EN 1998-1:2003 gives a constant amount of shear reinforcement irrespective of concrete strength. But the quantity is only about 950 mm^2 and is in proportion to the shear demand in the joint for exterior joint. With regard to NZS 3101:1995, the design shear reinforcement should be based on the values computed either by the condition as $A_{sh} \geq 0.4 V_{jh} / f'_{yh}$ or as per Eq. (23). In this particular example study, design shear reinforcement of 942 mm^2 has been obtained for 40% of the shear demand in the joint, which was governing rather than the values obtained as per Eq. (23). Hence in Fig.10, the required amount of steel remains constant and does not vary with respect to strength of concrete.

3.8.2 Vertical shear reinforcement

Vertical shear reinforcements basically sustain the truss mechanism. Besides, the vertical reinforcements resists vertical shear V_{jv}, and are provided in the form of intermediate column bars placed in the plane of bending between corner bars or vertical stirrup ties or special vertical bars, placed in the column adequately anchored to transmit required tensile force within the joint. In seismic design principles, column hinging is generally precluded and hence the stresses in column reinforcements are expected to remain within elastic range. Therefore, the vertical joint shear is expected not to be critical compared to horizontal joint shear. On this basis codes estimate the vertical shear reinforcement in proportion to the required horizontal shear reinforcement. Usually, since the intermediate column bars experience compressive stress less than yield stress, those bars as expected to have higher reserve strength to take tension due to vertical joint shear. Hence, it is acceptable to rely on column longitudinal distributed over the column face to take vertical shear along with flexural and axial load. ACI 318M-02 does not provide expressions for vertical shear reinforcement. However, the code insists on placement of intermediate column bars with restrictions on spacing on each face of the column. NZS 3101:1995 and EN 1998-1:2003 give specific recommendations to arrive at the necessary shear reinforcement in the vertical direction. As the vertical joint shear V_{jv} is expressed in proportion to horizontal joint shear, the vertical reinforcement is recommended to be arrived at from the horizontal shear reinforcement. The expression by NZS 3101:1995 is as follows:

$$A_{jv} = \alpha_v \frac{h_b}{h_c} A_{jh} \frac{f_{yh}}{f_{yv}} \tag{27}$$

where $$\alpha_v = \frac{0.7}{1 + \dfrac{N^*}{f'_c A_g}} \tag{28}$$

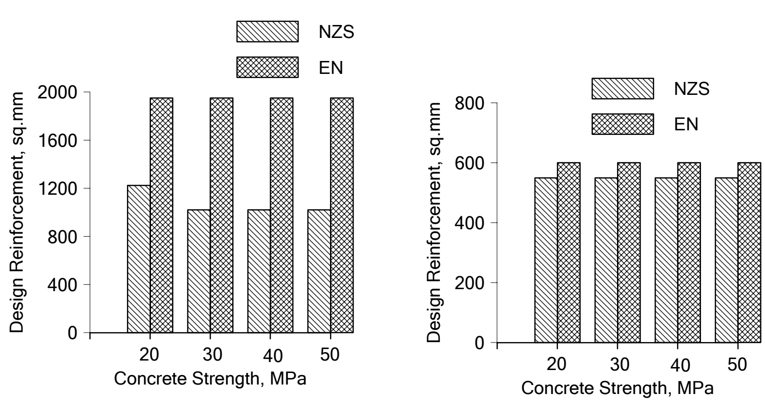
A_{jv} is total vertical shear reinforcement for the joint (not including corner vertical column bars); h_b is over all depth of beam; f_{yv} is lower characteristic yield strength of vertical shear reinforcement. The term α_v is included to account for the reserve strength in tension in the intermediate column bars. If the axial load on column is in tension, negative value is substituted to N^*. Consequently, the expression gives higher amount of reinforcement to be distributed as intermediate column bars.

Similarly, ENV 1998-1:2003 suggests the following expression

$$A_{sv,i} = \frac{2}{3} A_{sh} \left(h_{jc} / h_{jw} \right) \tag{29}$$

where $A_{sv,\,i}$, denotes the total area of the intermediate bars located in the relevant column faces between the corner bars of the column (including the bars that are part of the longitudinal reinforcement of the column). The code assumes that the intermediate column bars are subjected to compression approximately equal to 50% of their yield strength, thus offering a tensile stress margin of 1.5 $A_{sv,i}\, f_{yd}$ to take vertical shear.

The design shear reinforcement required for the vertical joint shear has been computed and compared for interior and exterior joints in Fig. 11 (a) and Fig. 11(b) respectively. ACI 318M-02 has not included any specific expression and hence not included in the comparison study. The values computed as per NZS 3101:1995 and EN 1998-1:2003 are plotted.



(a) Interior Joint (b) Exterior Joint

Fig.11 Vertical Design Shear Reinforcement

As per both codes, the vertical shear reinforcement required are obtained in proportion to horizontal shear reinforcement. EN 1998-1:2003 requires only 2/3 of the horizontal shear reinforcement as vertical reinforcement whereas NZS 3101:1995 adopts the reduction factor subjective to the axial load acting on the joint. In the example considered a compressive axial load ratio of 0.2 is used to arrive at the required reinforcement. The amount of vertical shear reinforcement arrived at as per NZS 3101:1995 is of about 0.58 times and as per EN 1998-1:2003 is of about 0.66 times the horizontal shear reinforcement.

3.9 Detailing for Shear Reinforcement

The shear reinforcement within the joint is provided in the form of closed stirrups, cross ties or hoops. The detailing requirements concern with the spacing and the arrangement of the hoops within the joint. The spacing requirements imposed by the three codes are summarized in Table.3. In principle, the restrictions on vertical spacing of transverse reinforcement are given with respect to the least member dimension to obtain adequate concrete confinement and in terms of diameter of longitudinal column bar to restrain buckling of bar after spalling of concrete. The spacing arrived from the above two criteria should be less than a prescribed numerical value. The spacing of horizontal lateral reinforcement is relaxed with respect to the confinement offered by the adjoining members and also based on the distribution of column longitudinal reinforcements. It can be seen that the spacing of lateral reinforcement is more or less the same for all the three codes of practice. As per NZS 3101 provisions, the spacing of the lateral reinforcement within the joint is relatively higher; nevertheless the code insists on adopting the spacing necessary to avoid buckling of column longitudinal bars within the joint.

Table 3 also gives the vertical spacing for the horizontal shear reinforcement provided in the form of stirrups and the horizontal spacing of vertical reinforcement, usually in the form of column longitudinal bars. If the column faces are confined on all four sides, ACI318M-02 allows the spacing to be relaxed up to 150 mm and EN 1998-1:2003 allows the spacing to be either equal to least core dimension b_o, or 150 mm whichever is minimum.

Table. 3 Spacing Requirements for Horizontal and Vertical Transverse Reinforcements, mm

Code	Vertical spacing for horizontal stirrups	Horizontal spacing of vertical reinforcement
ACI318M-02	min{ h_c/4 ; 6 d_b; s_x}	not more than 350
NZS 3101: 1995	min {10 d_b; 200}#	min {h_c/4; 200}
ENV 1998-1:2003	min {b_o/2; 8 d_b ;175}	150

Note: All dimensions are in mm

$s_x = 100 + \left(\frac{350 - h_x}{3} \right)$ where $100 < s_x < 150$

hx = max horizontal spacing of hoop or crosstie legs on all faces of the column

subject to variation with regard column ties requirement to avoid buckling

b_o minimum core dimension of the column

The preferred shape of a single leg cross-tie would have a 135-degree bend at both ends. Since installation with such a configuration is difficult, ACI 318M-02 allows standard 90-degree hook at one end of the cross tie with an extension not less than 6 times the diameter of the stirrup. But a 90-degree hook does not provide effective anchorage since it is not embedded in the confined column core. Hence, ACI 318M-02 recommends alternate placement of a 90-degree hook on opposite faces along the columns. However, in the case of exterior and corner connections, where the loss of cover could affect the anchorage of crossties at the 90-degree bend, it is recommended that only the 135-degree bend be used at the exterior face of the joint. However, NZS 3101:1995 and EN 1998-1:2003 prefer 135-degree bend at both ends. These two codes suggest an extension of 8 and 10 times the stirrup diameter respectively. Typical configurations are given in Fig. 12.

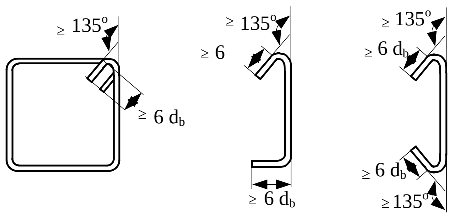


Fig. 12 Configurations of Stirrup and Cross-ties

All three codes recommend for necessary anchorage to be provided in the form of hooks or any other positive anchorage system in exterior joints, where the beam bar terminates at the joint core. The detailing principles are very same among the three codes. They allow the hooks to be bent towards the joint core. Hence, the top bars have to be bent down and the bottom bars should be bent up. The hooks have to be restrained with effective confinement with adequate horizontal hoops or stirrups.

4. SUMMARY AND CONCLUSIONS

The behaviour and expected performance of flexural members of reinforced concrete moment resisting frames can be realised only when the joints are strong enough to sustain the severe forces set up under lateral loads. Hence, the design and detailing of joints is critical, especially in seismic conditions. A comprehensive discussion of the issues and recommended procedures to be considered in the design of joints has been presented. The design aspects covered by ACI 318M-02, NZS 3101:1995 and EN 1998-1:2003 international codes of practice are appraised and compared.

The principles adopted in the design of joints by the three codes referred in this paper place high importance in providing for adequate anchorage of longitudinal bars and confinement of core concrete in resisting shear. The extent of variation in these provisions among the codes has been presented. The important observations from the comparison studies are enumerated below:

- ACI 318M-02 requires smaller column depth as compared to the other two codes for satisfying the anchorage conditions for interior and exterior joints. The effect of higher concrete grade in reducing the column depth has been included in EN 1998-1:2003 and NZS 3101:1995. The NZS 3101:1995 and EN 1998-1:2003 also account for column axial load in deciding minimum column depth from beam bar anchorage view point; however, the axial load effect on reducing the column dimension is only nominal. The requirement on the depth of column in interior joint is more compared to that in exterior joint.

- The criteria for minimum flexural strength of columns required to avoid soft storey mechanism is very stringent as per NZS 3101:1995 while the other two codes recommendations are comparable.

- The shear reinforcement required to ensure truss mechanism and to confine the core concrete varies considerably between the three codes. ACI 318M-02 requires transverse reinforcement in proportion to the strength of the concrete where as NZS 3101:1998 sets limits based on the level of nominal shear stress that is experienced by the joint core. EN 1998-1:2003 provides shear reinforcement to confine the joint and to bring down the maximum tensile stress to design value. Also, the code gives a bound on the estimate of shear reinforcement to maintain the integrity of joint after diagonal cracking. The design shear reinforcement is decided based on the above two criteria.

- NZS and EN code require 60% of horizontal shear reinforcement as vertical shear reinforcement. All three codes accept the intermediate column bars as a part of vertical shear reinforcement.

- The detailing requirements ensure adequate confinement of core concrete and preclude the buckling of longitudinal bar. The horizontal and vertical 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. NZS and EN codes emphasize on provision of 135^o hook on both ends of the cross-ties; whereas ACI code accepts 135^o at one end and 90^o hook at the other end and insists on proper placement of stirrups to provide effective confinement.

ACKNOWLEDGEMENT

This work has been supported through a project entitled "Review of Building Codes and Preparation of Commentary and Handbooks" awarded to the Indian Institute of Technology Kanpur by the Gujarat State Disaster Management Authority, Gandhinagar through World Bank finances. The views and opinions expressed therein are those of the authors and not necessarily those of the GSDMA or the World Bank. The authors are grateful to Professors Bhupinder Singh of NIT Jalandhar and A Meher Prasad of IIT Madras for their support and encouragement.

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LIST OF FIGURES

LIST OF TABLES

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Keywords:

Anchorage; beam-column joint; bond; code provisions; reinforced concrete frames; shear reinforcement.