This paper is the result of deliberations of the Society's discussion group on
SEISMIC DESIGN OF DUCTILE MOMENT RESISTING REINFORCED CONCRETE FRAMES

SECTION F

SHEAR STRENGTH REQUIREMENTS

T. Paulay*

F1.0 DESIGN FORCES

F1.1 - The design shear in members subjected primarily to flexure should be determined from considerations of static transverse forces on the member, with the flexural overcapacity being developed at the most probable location of critical sections within the member or in adjacent members, and the gravity load with the appropriate load factor.

F1.2 - The design shear force in members subjected to combined flexure and axial load should be determined from considerations of static forces on the member, with a rational adverse combination of the maximum probable end moments.

F2.0 WEB REINFORCEMENT

F2.1 - In determining the amount of transverse reinforcement in members subjected primarily to flexure the quantity v_c should be assumed to be zero for any seismic load combination in all regions where stirrup ties are required in accordance with Section E.

F2.2 - In members subjected to combined flexure and axial load the web reinforcement should be at least as much as required by Sections 11.4 and 11.6 of ACI 318-71, except that in the end regions, adjacent to the face of the intersecting member, the following provisions should apply:

(a) The end region of a member should be assumed to extend a distance away from the face of the intersecting member not less than: (1) the depth of the member in the plane in which the frame resists the earthquake forces under consideration, (2) one-sixth of the clear span or height of the member, or (3) 450mm.

(b) In the end regions the quantity v_c should be assumed to be zero unless the design axial compression force produces an average stress in excess of $0.10f_c'$ over the gross concrete area.

(c) When the average compression stress in the member exceeds $0.10f_c'$ the value of v_c should be computed by:

$$v_c = 0.25\left(1 + \frac{f_c'}{25}\right)\sqrt{\frac{N_u}{A_g} - \frac{f_c'}{10}} \qquad \text{(F.1)}$$

F2.3 - At regions extending to a distance of not less than d from the face of the supports of flexural members, where due to reversed lateral load the top and bottom flexural reinforcement may be subjected to tensile yielding, and at any other similarly loaded potential plastic hinge region, the value of v_u should not exceed

$$v_u = \frac{(2 + r)\sqrt{f_c'}}{4} \qquad \text{(F.2)}$$

where r is the ratio of the maximum values of the shear force developed with positive moment hinging to the shear force developed with negative moment hinging, taking into account the sense of the shear forces, unless, in addition to the transverse reinforcement requirements of Section E, the following conditions are satisfied at every section taken at right angles to the axis of the member:

(a) At least 75% of the corresponding shear in the regions is resisted by diagonal reinforcement across the web.

(b) The entire shear in the regions is resisted by diagonal reinforcement across the web when the value of v_u exceeds one and one half times that given by Eq. (F.2).

The value of r should not be taken larger than zero nor less than minus one.

F2.4 - Rational analysis must show that the shear resistance, in accordance with F2.3(a) and (b), at each cross section of the potential plastic hinge zone is provided by the transverse component of the inclined steel forces only. When diagonal bars cross a section in two directions the transverse components of the diagonal tension and compression steel forces may be considered together.

F2.5 - The requirements of Section F2.3 do not apply to members designed in accordance with Section F2.2 unless the minimum axial compression stress on the gross concrete area, associated with the maximum shear on the member, is less than $0.10f_c'$.

F2.6 - The value of v_u in a member should not exceed $0.8\sqrt{f_c'}$ unless the member is fully designed as a diagonally reinforced beam.

F3.0 OPENINGS IN THE WEB

F3.1 - Adjacent openings for services in the web of flexural members should be arranged so that potential failure planes across such openings cannot occur.

F3.2 - Small square or circular openings may be placed in the mid-depth of the web provided that cover requirements to longitudinal and transverse reinforcement are satisfied, and the clear distance between such openings, measured along the member, is not less than 150mm. The size of small openings should not exceed 1000 square mm for members with an effective depth less than 500mm, or $0.004d^2$ square mm when the effective depth is more

* Professor of Civil Engineering, University of Canterbury.

than 500mm.

F3.3 - Webs with openings larger than that permitted by F3.2 should be subject to rational design to ensure that the forces and moments are adequately transferred in the vicinity of the openings.

F3.4 - Whenever the largest dimension of an opening exceeds one quarter of the effective depth of the member it is to be considered large. Such openings should not be placed in the web where they could affect the flexural or shear capacity of the member, nor where the design shear stress exceeds $0.4\sqrt{f'_c}$, or in potential plastic hinge zones. In no case should the height of the opening exceed 0.4d nor should its edge be closer than 0.33d to the compression face of the member.

F3.5 - For openings defined by F3.4 longitudinal and transverse reinforcement should be placed in the compression side of the web at one side of the opening to resist one and one half times the shear and moment generated by the shear across the opening. Shear transfer by part of the web on the tension side of the opening should be neglected.

F3.6 - Transverse web reinforcement, extending over the full depth of the web, should be placed adjacent to a large opening over a distance not exceeding one half of the effective depth of the member to resist twice the entire design shear across the opening.

COMMENTARY

CF.1 DESIGN FORCES

CF1.1 - This section is intended to deal with beams. Typically two plastic hinges may form in a beam, such as at A and B in the span shown in Fig. C.F.1. With the corresponding flexural overcapacities, in accordance with the definitions, denoted as M_{OA} and M_{OB}, the design shear force at B will be

$$V_{UB} = \frac{M'_{OA} + M_{OB}}{l_{AB}} + V_{DB} + 1.3V_{L_RB}$$

where V_{DB} and V_{L_RB} are the reactions for the simply supported beam A - B at B due to dead (D) load and reduced live (L_R) load, in accordance with NZS 4203:1976. Similarly the critical shear for the same beam at A will be

$$V_{UA} = \frac{M_{OA} + M'_B}{l_{AB}} + V_{DA} + 1.3V_{L_RA}$$

$$= \frac{M_{OA} + M_{OC}}{l_{AC}} + V_{DA} + 1.3V_{L_RA}$$

where V_{DA} and V_{L_RA} are the dead and reduced live load reactions respectively for the length l_{AB}. The value of M'_B must be evaluated from the overstrength capacity in the vicinity of C. It will be noted that the shear at C for this load combination is zero. It is the intent to prevent a shear failure under maximum possible lateral forces. Accordingly the dependable shear strength must be equal or larger than the shear obtained above.

CF1.2 - This section gives guidance in general terms for the determination of shear forces across columns subjected to bending with compression or with tension. With few exceptions the shear results from the applied end moments due to lateral load at the top and the bottom of the column. The determination of these moments is uncertain because of the dynamic and random nature of the loading. The intent of capacity design, when applied to columns, is to reduce the likelihood of column yielding and to prevent a shear failure under the maximum possible lateral forces. When earthquake effects are determined from equivalent lateral static forces, in accordance with NZS 4203:1976, a procedure given in Section G may be used to determine the design shear forces.

CF2.0 WEB REINFORCEMENT

CF2.1 - In conformity with similar recommendations in codes of other countries, it is assumed that the contribution of the concrete to shear strength is negligible in plastic hinge zones and hence web reinforcement is required for the full shear demand. The plastic hinge zones have been defined in Section E. In between these regions the value of v_c may be taken as specified in ACI 318-71 Chapter 11 and the web reinforcement provided according to Section 11.6 of that code. If v_c is to be zero in beams because of one seismic load combination it is to remain zero for any other load. For example a positive plastic hinge in a beam away from a column face is subjected to no shear. However, extensive yielding will reduce its shear capacity for any other load combination that produces shear in this location of the beam without causing significant moments.

CF2.2 - In columns subjected to axial load and moment the requirements of Sections 11.1, 11.2, 11.4 and 11.6 of ACI 318-71 apply except in the end regions adjacent to intersecting members, i.e. beams. Yielding or hinging in a column, if it is to occur, is assumed to be confined to the end regions. Therefore the middle portion of a column may be assumed to remain elastic and thus the contribution of the concrete to shear strength, v_c, may be assumed to be sustained under the most adverse seismic conditions. In this portion the stirrup spacing should not exceed 0.4 times the depth of the column in accordance with Section 11.2 of ACI 318-71.

The end regions, representing the localities of potential plastic hinges, have been defined the same way as the regions at which confining column reinforcement is required in accordance with Section H. For small axial compression, i.e. $N_u/f'_cA_g < 0.10$ the value of v_c is to be zero as for beams without axial load. For larger axial compression Eq. 11.6 of ACI 318-71 has been modified to give a gradual rather than an abrupt increase in v_c (Eq. F.1). For values of $N_u/f'_cA_g > 0.3$ the values of v_c are very close to those given by Eq. (11.6), as can be seen in Fig. CF.2, recognising the contribution of the concrete to shear strength with increasing axial compression, even in potential plastic hinge zones.

When computing v_u, v_c and N_u it is necessary to ensure that they correspond to the same seismic load combination. For the magnitude of N_u in Eq. (F.1) the minimum probable value consistent with the acting shear force, should be assumed.

CF2.3 - These provisions have been made to safeguard beams, subjected to reversed cyclic loading, against sliding shear failure and to reduce the loss of energy dissipation due to slip in the plastic hinge zones. When the top and bottom flexural reinforcement progressively yield, wide full depth cracks will develop. This may significantly reduce the interface shear transfer capacity of the concrete and a detrimental overloading of the dowel mechanism of the longitudinal flexural reinforcement may ensue. Therefore diagonal reinforcement needs to be provided at every section taken at right angles across the plastic hinge zone, effectively crossing potential full depth cracks, when the nominal shear stress in both directions exceeds $0.25\sqrt{f'_c}$. When the shear stress due to gravity load on a beam is significant, the combined gravity and seismic shear at a positive moment hinge near a column face may be zero or very small. Such is the case at support A of the beam shown in Fig. CF.1. In this case shear reversal at that plastic hinge does not occur and hence no grinding of the concrete along wide full depth cracks is to be expected. Consequently the algebraic value of the ratio r of the shear forces developed during positive and negative hinge formation is to be taken as zero and the maximum value of v_u, obtained from Eq. (F.2) will be $0.5\sqrt{f'_c}$, before diagonal web reinforcement is required to control sliding.

When the nominal reversable shear stress v_u at the plastic hinge zone is large the entire shear must be resisted by diagonal reinforcement to prevent sliding. According to F.2.3(b) the limiting stress will be one and a half times that given by Eq. (F.2). Depending on the relative magnitude of the reversed shear, the limits when diagonal shear reinforcement is required will be $0.375 < v_u/\sqrt{f'_c} < 0.75$.

When according to F.2.3.(a) the diagonal web steel across the potential plastic hinge zone does not resist the total shear, the available stirrup ties must have a minimum capacity of resisting the remainder of the shear force. Such a case is shown in Fig. CF.3(a).

These provisions do not affect plastic hinge zones in the positive moment area away from the face of the beam support where, as at C in Fig. C.F.1, shear stresses will be low and the top reinforcement is not likely to be subject to tensile yielding.

CF2.4 - The primary purpose of diagonal web reinforcement in this case is not to form part of the traditional truss mechanism, but to effectively cross every potential full depth crack after the flexural reinforcement in both faces of a member has yielded. A rational analysis is required to show that the vertical component of the diagonal web reinforcement across each section of the potential plastic hinge within a distance d away from the theoretical section of maximum moment, such as a column face, is equal or larger than the shear force to be resisted.

The area of the diagonal reinforcement A_{vd} required to resist a shear force V_s at a potential full depth crack can be computed from

$$A_{vd} = \frac{V_s}{\phi f_y \sin\alpha}$$

where $V_s = 0.75 v_u b_w d$ according to F.2.3.(a)

and $V_s = 1.00 v_u b_w d$ according to F.2.3.(b)

It will be more expedient to utilise an existing bar or a suitable bar diameter in the locality and to determine the inclination of the bar, α, to the longitudinal axis of the member.

The inclination of bent bars with respect to the longitudinal axis of the member should not be less than 30° nor should it be more than 60°. Such bars must be adequately anchored so as to develop their strength at every part of their inclined length.

When inclined bars are required to resist the shear in both directions in a beam subjected to large earthquake induced shear forces and relatively small gravity shears, the vertical components of both the inclined tension and compression bars may be assumed to contribute to the total shear resistance across every cross section of a potential plastic hinge zone.

When computing the shear strength of the plastic hinge region along a potential 45° failure plane, only the contribution of the diagonal tension reinforcement may be added to the resistance of stirrups. The interpretation of the requirements, intended to prevent diagonal tension and sliding shear failure in a potential plastic hinge region, that is removed from a column force, is shown in Fig. CF.3(b) for the case when reversed shear forces of equal intensity are expected.

In the evaluation of the flexural overstrength capacity of a plastic hinge, the contribution of the diagonal reinforcement to the development of moment should not be overlooked.

CF2.5 - Plastic hinges, if any, that may develop in columns which are designed in accordance with these recommendations, are not expected to be exposed to large ductility demands and hence to reversed cyclic tensile yielding. Moreover, at least for one direction considerable axial compression will be present that would close wide cracks. Also the dowel action of a considerable number of the longitudinal column bars is likely to be more effective in the confined end regions. For this reason the requirements of Section F.2.3 need not be applied to members referred to in Section F2.2. However, if the axial compression is low and it does not exceed an average stress of $N_u/A_g = 0.10f'_c$ for a particular direction of lateral load, the column should be treated as a beam. Such columns may occur in low buildings or in the upper storeys of multistorey frames. In most cases it is likely that the shear stress in such columns will not exceed $0.25\sqrt{f'_c}$ MPa.

Columns subjected to moment and axial tension should be treated as beams in accordance with Section F2.3. Thus diagonal shear reinforcement should be provided across the critical end region sections of such a column when the nominal design shear stress exceeds $0.25\sqrt{f'_c}$ MPa. It should be noted that for this situation it is intended to allow a hinge formation and hence considerable yielding of the column steel. With large axial tension the neutral axis depth may be so small that very little if any concrete will be in contact at the critical section. The recommendation implies that the equivalent of up to $0.25\sqrt{f'_c}$ stress can be carried by dowel action only.

CF2.6 - The maximum nominal shear stress is limited to $0.8\sqrt{f'_c}$. This is a little less than that permitted by Section 11.6.4 of ACI 318-71 in regions where the contribution of the concrete, v_c, can be taken into account.

When diagonal reinforcement is provided in such a way that the entire moment and shear force at every section can be resisted by steel forces in tension and compression only, as in the case of diagonally reinforced coupling beams, there appears to be no need to limit the nominal shear stress v_u, which under these circumstances is an irrelevant index.

CF3.0 OPENINGS IN THE WEB

These recommendations have been largely restricted to the restatement of general principles in "good engineering practice". There is relatively little in the literature that is relevant to seismic load conditions, for which in principle, more stringent rules should apply.

CF3.1 - Openings must be located in such a way that no potential failure planes, passing through several openings, can develop. In considering this the possible reversal of the shear forces, associated with the development of the flexural overcapacity of the members, should be taken into account.

CF3.2 - Small openings with areas not exceeding those specified are considered not to interfere with the development of the strength of the member. However, such openings must not encroach into the flexural compression zone of the member. Therefore the edge of a small opening should be no closer than 0.33d to the compression face of the member, as required by F3.4. When two or more small openings are placed transversely in the web above each other the distance between the outermost edges of the small openings should be considered as being equivalent to the height of one large opening and the member should be designed accordingly.

CF3.3 - Parts of the web adjacent to an opening, larger than that permitted by F3.2, should be subjected to rational analysis to ensure that failure of the member at the opening cannot occur under the most adverse load conditions. This will require the design of orthogonal or diagonal reinforcement around such openings.

CF3.4 - More severe restrictions apply when the largest dimension of an opening in the web exceeds 0.25d. Openings of this size are not permitted in areas of the member where the design shear stress exceeds $0.4\sqrt{f'_c}$ nor in plastic hinge zones. To ensure that the moments and shear forces can be effectively transmitted by the compression zone of the member, the opening must not encroach into the flexural compression zone. The dimension of the opening at right angles to the axis of the member must not exceed 0.4d. The horizontal clear distance between adjacent large openings in a beam should not be less than twice the length of the opening or the depth of the member, whichever is more.

CF3.5 - Only the part of the web above or below an opening which is in compression should be considered to transmit shear. The stiffness of the tension part, which accommodates the longitudinal flexural reinforcement, is considered to be negligible because of extensive cracking. The amount, location and anchorage of the longitudinal reinforcement in the compression part of the web above or below the opening must be determined from first principles so as to resist one and one half times the moment induced by the shear force across the opening. Similarly shear reinforcement in the compression chord, adjacent to the opening, must resist 150% of the design shear force. This is to ensure that no failure should occur as a result of the local weakening of the member by the opening. Effective diagonal reinforcement above or below the opening, resisting one and one half times the shear and moment, is also acceptable.

CF3.6 - At either side of an opening where the moments and shear forces are introduced to the full section of a beam, horizontal splitting or diagonal tension cracks are to be expected. To control these cracks transverse reinforcement, resisting at least twice the design shear force, should be provided on both sides of the opening. Such stirrups can be distributed over a length not exceeding 0.5d at either side immediately adjacent to the opening.

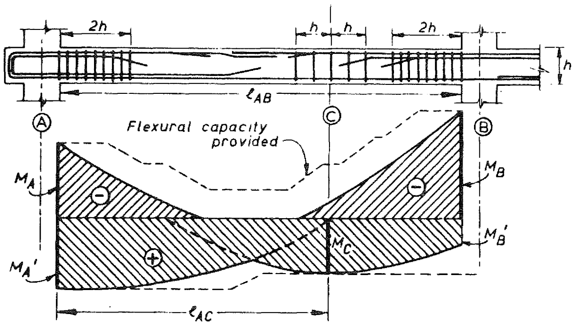


FIGURE C.F.1: LOCALITIES OF POTENTIAL HINGES WHERE STIRRUP TIES ARE REQUIRED.

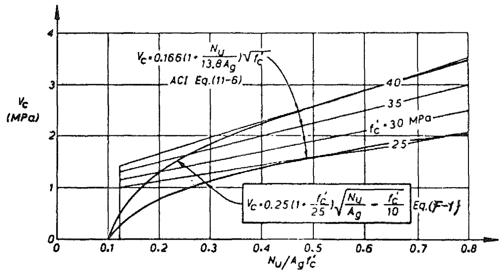
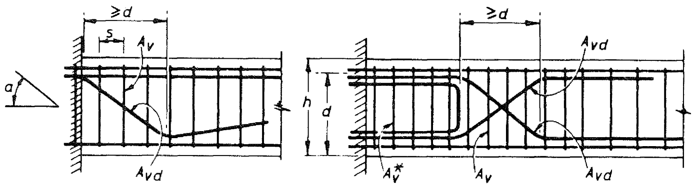


FIGURE C.F.2: THE SHEAR STRENGTH OF CONCRETE IN COLUMNS.



$r = -0.5$ | $\phi = 0.85$ | $r = -1.0$

F.2.3(a)

$$0.375 < \frac{v_u}{\sqrt{f'_c}} < 0.562$$

(i) For sliding shear

$$A_{vd} > \frac{0.75\ v_u b_w d}{0.85\ f_y \sin\alpha}$$

(ii) For diagonal tension

$$A_v > \frac{0.50\ v_u b_w s}{0.85\ f_y}$$

because of the reversed shear force

Case (a)

The upward shear is 50% of the downward shear and section F.2.3(a) applies.

F.2.3(a)

$$0.25 < \frac{v_u}{\sqrt{f'_c}} < 0.375$$

(i) For sliding shear

$$A_{vd} > \frac{0.75\ v_u b_w d}{0.85\ f_y 2\sin\alpha}$$

(ii) For diagonal tension

$$A_v > \frac{0.625\ v_u b_w s}{0.85\ f_y}$$

F.2.3(b)

$$0.375 < v_u < 0.8$$

$$> \frac{1.0 v_u b_w d}{0.85\ f_y 2\sin\alpha}$$

$$> \frac{0.50\ v_u b_w d}{0.85\ f_y}$$

Case (b)

Reversed shear of equal magnitude acts across a plastic hinge away from a column face. The relevant design equations corresponding with different shear stress levels in accordance with sections F.2.3(a) and F.2.3.(b) are given.

FIGURE C.F.3: EXAMPLES FOR THE DESIGN OF DIAGONAL SHEAR REINFORCEMENT IN POTENTIAL PLASTIC HINGE ZONES TO CONTROL SLIDING AND DIAGONAL TENSION FAILURES