based on these tests, requires a horizontal force equal to $0.2V_u$ and provides a complex equation for the allowable shear, at the same time limiting the ρ to be used in this equation. The Commentary also suggests that the bracket plate be welded to the top bars where such tension is possible. Where provision is made to avoid the horizontal loading, the allowable shear may be simplified to (Code 11.14.3)

$$v_u = 6.5(1 - 0.5a/d)(1 + 64\ \rho_v)\sqrt{f_c'}$$

where ρ_v is based on the sum of the flexural steel plus lower horizontal ties of area A_{vh} (equal at least to $0.5A_s$ and distributed over the upper two-thirds of the effective depth) and is limited in this equation to $0.20f_y/f_c'$. The Code limits the effective depth d used at the column face to twice the depth at the outside edge of the bearing and the Commentary suggests that this bearing not start closer than 2 in. from the bracket face. The extension of the tension bars as close to the face as possible and an anchor provided by a welded cross bar of the same diameter is also recommended (Fig. 4.22*c*).

The problem area in a long bracket is the anchorage of bars because the variable depth (inclined strut idea) makes the bar tension nearly as critical at the load as at the column face.

If the anchorage can be extended as in Fig. 4.22*b*, the shear recommendations made in the second edition of this text and shown in Fig. 4.22*a* are still valid[9] in the author's opinion; they are simpler and a little less restrictive. For the longer brackets the author would normally calculate the required A_s at the face of column, keep the bars of a diameter that can be anchored (or try for a deeper bracket if necessary to make them smaller), and if a horizontal tension pull were expected add enough bars to resist it. Finally the shear should be checked and should rarely control. For a short bracket the shear may be the chief control on the bracket depth. For a heavy bracket, say more than 18 in. deep, additional horizontal U-bars or ties in the upper half will avoid wide cracks, add to the bracket strength, and even help with the moment. Vertical stirrups are essentially useless. The Code requirement is horizontal steel A_h equal to half the top A_s.

4.17 SHEAR-FRICTION FOR BRACKETS

The shear-friction theory (Code 11.15) is particularly useful for precast assemblies and composite construction.[10,11] The Code suggests this approach may be used for brackets cast-in-place (with the same reinforcement limitations of Code 11.14 provided $a/d \ngtr 0.5$.

The shear-friction approach is to assume a shear plane already cracked as in Fig. 4.22*d*, with a coefficient of friction μ given in Code 11.15.4 (1.4 if cast monolithically). The depth must then be such that the shear stress on the area *bd* will not exceed $0.2f_c'$ or 800 psi. The necessary A_{vf} must be large enough to resist the normal force necessary to support the shear by friction:

$$A_{vf}f_y = \mu(V_u/\phi)$$

Finally A_{vf} is checked to see that it lies between $0.5A_s$ (Code 11.14.4) and $1.0A_s$ (Code 11.14.3), where A_s is the normal reinforcement for flexure plus that used for any horizontal component of load. Closed stirrups or ties are appropriate for A_{vf} and these should be distributed uniformly within two-thirds of d adjacent to A_s.

4.18 DEEP BEAMS

(a) General Case

Deep beams in Code 11.9 are totally new in the Code. Although the Code deals only with shear, a few general ideas first might be helpful. A deep beam is simply a member short enough to make shear deformations important in comparison to pure flexure; the Code considers a clear span l_n of up to $5d$. Plane sections in these beams do not remain essentially plane under loading; but this is also true at the end of every simple span beam. If one has a clear picture of how any beam fails in shear at $a/d = 1$, he can get a rough idea of how a deep beam of $l_n = 2d$ would fail in shear by imagining two such end sections of beam joined together with the load at the junction. At $a/d = 1$ in a long beam, flexure will not be of much interest; in the deep beam of $l_n = 2d$, flexure at the load is the critical flexure. If the load is applied to the top of the deep beam the lever arm z of the internal couple will be smaller than usual, say $0.8d$; if the load is applied to the bottom of the beam in tension, z may drop as low as $0.4d$ or $0.5d$. The Code provisions are limited to loading on top of the beam. Anchorage of tension steel becomes critical. Leonhardt[12,15] suggests *horizontal* hooks on tension bars (vertical compression helping to avoid splitting) or U-shaped bars lap spliced at midspan.

If such a deep beam is provided with stirrups that are able to deliver the bottom load to the upper part of the beam, the beam will behave nearly like a top loaded beam. It is always desirable in any kind of beam to provide hangers (stirrups) to pick up bottom loads in this fashion. Experimental evidence indicates that stirrups that must act as hangers *and* as web reinforcement need *not* be designed for the sum of the two requirements, but simply for the larger of the two.