Tension forces in the beam end truss model, Fig. 2, are carried by deformed reinforcing bars meeting ASTM A615 or A706, both Grade 60 yield strength. The A706 low-alloy steel reinforcing bars are used for the "hanger rebar steel", shown as Forces TF1 and HF1 in Fig. 2. The A706 bars are used for welding some of the TF1 bars to the BSF unit front plate, and for small diameter bending of the other TF1 bars to have direct contact on the BSF unit front end top saddle. The bending diameters are less than specified by ACI 318, but in accordance with ASTM.

The column unit requires reinforcing to provide anchorage for the eccentric bearing load (knife plate) acting on the column box, anchorage of knife horizontal sliding force (beam axial force), and also to provide a reinforced bearing below the column box to permit higher concrete bearing stresses. The reinforced bearing is designed in accordance with the PCI Design Handbook using shear-friction theory.

The beam/column reinforcing design examples included in this reference show example material and fabrication requirements for precast concrete beams and columns with BSF connections. Design strengths are shown for each example. The design strength is the design (ultimate) strength capacity (ϕRn) calculated in accordance with the ACI-318 Building Code. Connection design strength is required to equal or exceed the required strengths calculated for the factored load combinations stipulated in the ACI 318 Code.

NORWEGIAN RESEARCH AND FULL-SCALE TESTING

Product development of the BSF connections involved two series of load tests. The original BSF connections were load tested by the Norwegian Building Research Institute, Oslo, Norway, in 1988. An internal truss calculation model for design of the force transfer from the steel box unit to the concrete beam was developed and experimentally documented through full-scale tests at SINTEF Structures and Concrete, Trondheim, Norway in 1992.

Both series of load tests consisted of applying axial tension and vertical loads to concrete beams containing BSF end connections. The axial tension loads in the test were 40 to 50 percent of the vertical test loads, and represented shrinkage, creep and temperature contraction forces by sliding (friction) of the knife plate within the column box.

The first series of 51 load tests, conducted in 1988, documented the capacity of the original BSF connection units. The internal truss calculation model was developed from the first test series, and verified in 14 additional load tests carried out in 1992. This test series included measuring strains in the reinforcement at about 20 points in each test specimen.

The second test series documented the theoretical truss models reasonably well, and clarified which type of reinforcement provided the best structural behavior of the connections. The current version of the BSF connections and reinforcement were developed from the test results, including the following changes to the beam units:

1. Top plate replaced by half-round steel profile or "saddle" (Fig. 1(c)(d) Part 7) to provide direct contact with front hanger rebar steel. (Fig. 2, Force TF1)

2. Use of a front plate flush with the end of the beam and welded to the beam box side plates and top half-round saddle. (Fig. 1(c)(d) Part 8)

3. The bottom plate reduced in width to be the same as the beam box (side plates). The bottom plate in the smaller units is replaced by the longitudinal reinforcing bar anchoring the unit for axial tension loads.

The simplified truss model for design, shown in Fig. 2, was developed by Partek Ostspenn from the more rigorous theoretical models documented in the second load test series. The second load test series results showed the beam ends with BSF connections had greater ultimate capacity than predicted by this simplified truss model. The simplified truss calculation model neglects the additional strength contribution of: (1) the normal beam end shear reinforcement used around the BSF beam box unit; and (2) the cantilever moment capacity of the steel beam box and bottom longitudinal reinforcing bar welded to the beam box.

Some of the SINTEF test results are presented in Tables 1 and 2. The results indicate the maximum loads achieved during the tests easily exceeded the ultimate capacity calculated using the hanger rebar steel (Fig. 2, Force TF1) yield capacity and the ACI 318 Code concrete shear strength limits. Figures 3 and 4 show reinforcing arrangement, strain gage locations and crack pattern after fracture for test beams B4A and B7B. The ACI 318 Code limits nominal concrete beam shear stress to $10\sqrt{f'_c}$, regardless of the amount of shear reinforcement, to conservatively control diagonal truss mechanism concrete stresses to a value below the crushing strength of the concrete.

The BSF units used for the five tests presented in Tables 1 and 2, except for Test B5A, did not contain a vertical front plate (Fig. 1(c)(d) Part 8) welded to the beam box side plates and half-round top saddle. Also, the hanger rebar steel in Test B6A, and a portion of the hanger rebar steel in Test B7B consisted of closed vertical stirrup bars NOT bent into horizontal bars at the beam bottom to directly resist truss analogy tension Force HF1 (Fig. 2). Figure 4 shows the stirrup type TF1 hanger rebar steel and the combined TF1 and HF1 hanger rebar steel used in Test Unit B7B. Stirrup type vertical hanger steel requires the partially developed straight end of the main beam bottom flexural reinforcement to resist the HF1 tension force.

Detail test results are available in the report "The BSF System - Calculation Model and Experimental Investigation", SINTEF Structures and Concrete, N-7034, Trondheim, Norway, December 1992 (Report Number STF70 F92150).

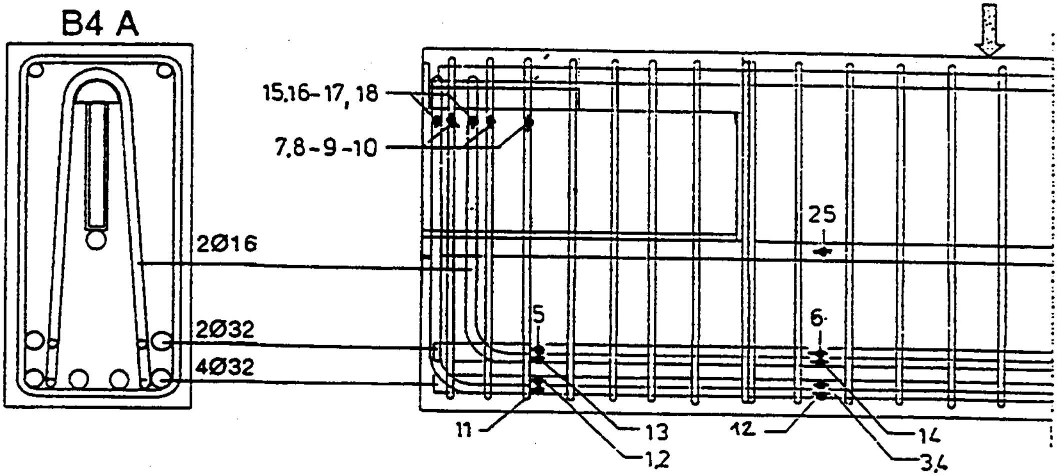
Table 1 - BSF Test Results - Partial Summary

SINTEF Test	Test Beam Width (in.)	Test Beam Height (in.)	f 'c (psi) (1)	Max. Vert. Test Load (kips) (2)	Max. Test Beam Shear Force (kips) (3)	Max. Test Beam Net Shear (psi) (4)	ACI Shear Limit (psi) $10\sqrt{f'c}$ (5)	TF1 Reinf. Capacity (kips) (6)
B4A	11.8	23.6	7370	151	196	972	858	50
B5A	7.9	19.7	6640	119	155	1570	815	100
B5B	7.9	19.7	6640	98	127	1293	815	50
B6A	7.9	19.7	7120	117	152	1544	844	85
B7B	11.8	19.7	7050	151	196	1199	840	135

(1) Actual test beam 28 day concrete cylinder strength
(2) Maximum vertical test reaction applied to BSF knife plate (Fv)
(3) Approximate maximum vertical shear force in test beam between Figure 2 truss analogy forces TF1 and TF2 (vertical shear along concrete strut CF zone); magnitude equals hanger force TF1
(4) Approximate maximum concrete shear stress using shear area based on deducting width of beam box, and depth to C.G. of hanger steel bottom horizontal reinforcement (Figure 2 force HF1): Av = (Beam Width - 2 in.) x (Beam Depth - 3 in.)
(5) ACI nominal concrete shear limit for beams, using test actual concrete strength
(6) Yield capacity of vertical reinforcement in direct contact with steel half-round on top of BSF beam box, using actual reinforcement Fy = 80.6 ksi; Compare to Max. Shear Force(3), which equals hanger force TF1

Table 2 - BSF Test Results - Partial Summary

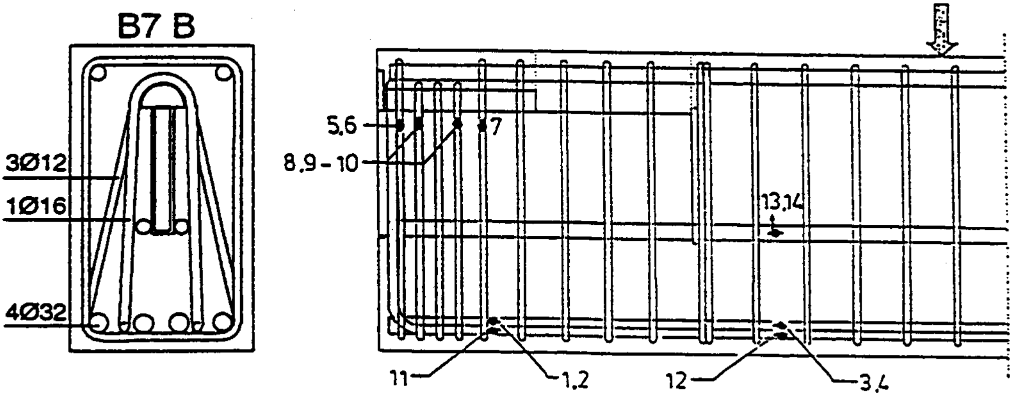
SINTEF Test	Type of Failure
B4A	Steel box failure, test stopped; half-round weld failure at 90 kips
B5A	Concrete shear - tension
B5B	Concrete shear / bond to bottom of BSF beam box
B6A	Push-out of concrete; half-round weld failure at 67 kips; main bot. reinforcement bond failure
B7B	Push-out of concrete; half-round weld failure



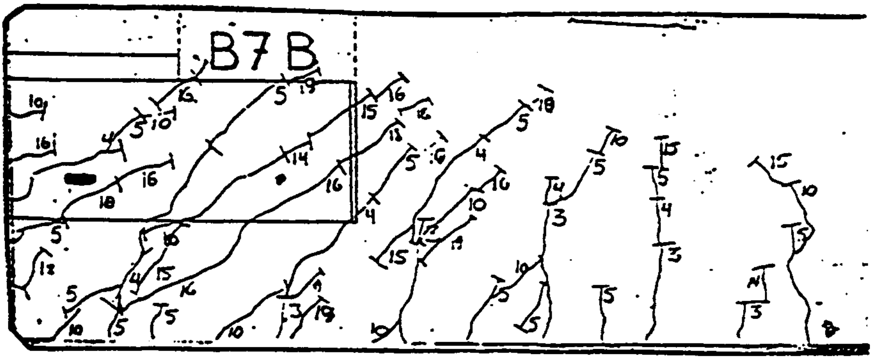
Location of Reinforcement and Strain Gages

Crack Pattern After Failure

Fig. 3 SINTEF Test Connection B4A



Location of Reinforcement and Strain Gages



Crack Pattern After Failure

Fig. 4 SINTEF Test Connection B7B