BSF PRECAST CONCRETE BEAM TO COLUMN CONNECTION SYSTEM

"THE INVISIBLE CORBEL"

TECHNICAL DATA

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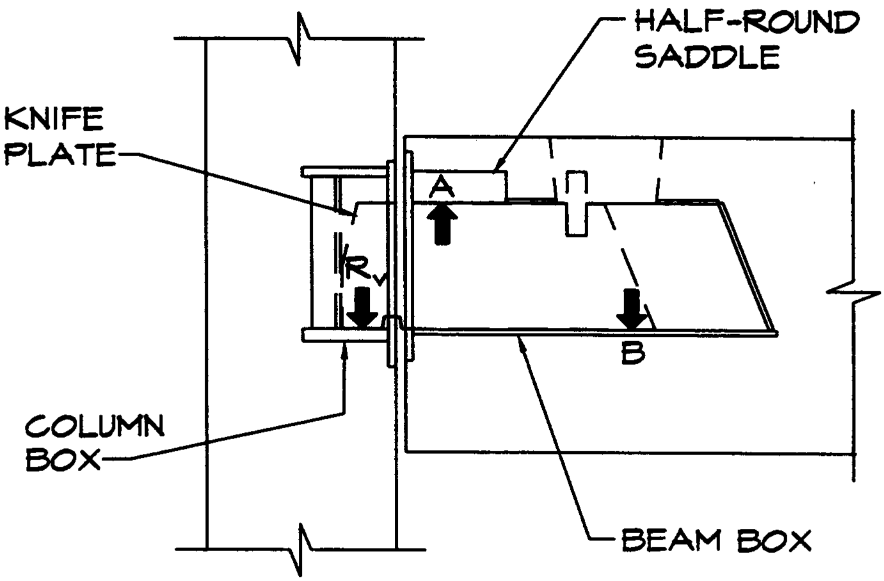
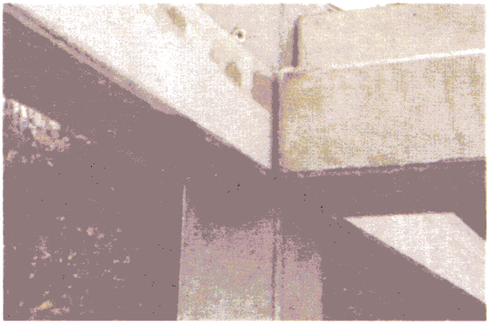
PART 1

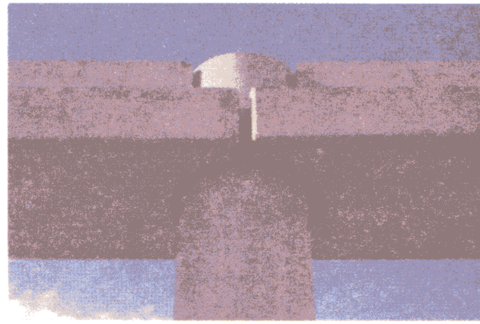
INTRODUCTION

The BSF connection is a hidden beam end connection for gravity loads that eliminates the need for projecting column corbels. It provides a simple, efficient connection that allows the designer new freedom in creating clean, elegant lines in the completed precast/prestressed concrete structure. The BSF connection can be used in all types of building structures where beams frame into columns, such as office buildings, hotels, parking structures, schools, sports stadiums, and other similar structures.



From a steel box cast into the concrete beam end, a sliding steel "knife" plate with a safety notch is cantilevered into a steel box that has been cast into the concrete column. The efficient "invisible corbel" allows new possibilities in geometrical design with greater freedom for architectural design of precast concrete structures.





HISTORY

The BSF connection system was developed in Norway in 1987 by Partek-Ostspenn and has been successfully used in Europe (Norway, Finland, Holland, Germany, Great Britain and other countries) with over 15,000 connections during the first 5 years of use. The original BSF connections were load tested by the Norwegian Building Research Institute, Olso in 1988. A 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 in 1992. A new version of the BSF connection system was introduced in 1993 with reduced costs, increased capacities, and wider application.

DESCRIPTION

The BSF connection consists of a fabricated steel box unit cast into the beam end, a steel box unit cast into the support column, and a sliding steel "knife" plate within the beam box unit. When the beam is hoisted into final position, the knife is pushed out of the beam into the column box unit with a special tool. Beam end joint width adjustments are made with the same tool, without the use of the hoisting equipment. The BSF connection requires no screws, bolts, or welding at the site, and is consequently safe and fast during erection. Tolerances allowed in the system have proven to be reasonable and workable within accepted industry standards.

- **PRACTICAL ADVANTAGES**

The value of the BSF connection system over typical projecting column corbels in precast structures include:

- Six BSF connection sizes with standardized hardware and published load capacities that reduce chances for design and production errors.

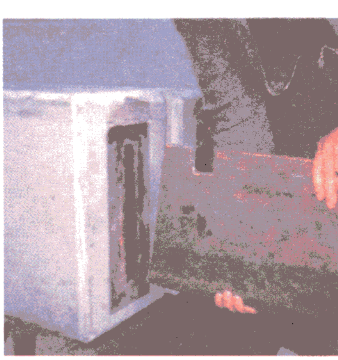
- Cost savings and efficiency in beam and column fabrication and handling.

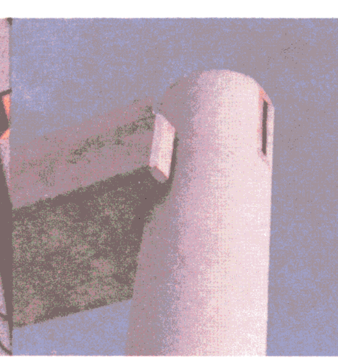
- Uncomplicated and fast erection.

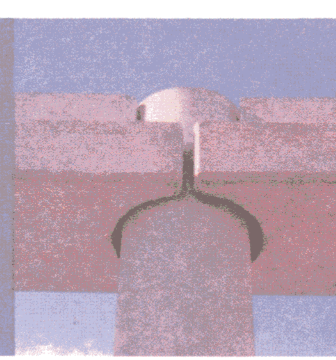
- Functions within normal building tolerances.

- Allows for axial volume change movements due to creep, shrinkage, and temperature change.

- Easily protected against fire or corrosive elements.







- **BSF STANDARDIZED SIZES**

The following six standard BSF connection sizes have been developed to provide a range of load capacities to meet the needs of the precast/ prestressed building industry.

VERTICAL DESIGN (ULTIMATE) STRENGTH (ϕR_n)

	KNIFE PLATE	AISC (LRFD) (1)	ACI 318 (2)	MIN. BEAM SIZES (3) (f'c = 5000 psi)	
BSF TYPE	SIZE (IN. X IN.)	KIPS (kN)	KIPS (kN)	WIDTH (IN)	HEIGHT (IN)
150/20	5.91 X 0.79	45 (200)	50 (222)	8	22
200/20	7.87 X 0.79	67 (300)	75 (334)	10	24
200/30	7.87 X 1.18	101 (450)	110 (489)	14	24
200/40	7.87 X 1.57	135 (600)	150 (667)	15	30
200/50	7.87 X 1.97	157 (700)	175 (778)	16	34
250/50	9.84 x 1.97	213 (945)	235 (1050)	21	36

(1) AISC design strength shown must be equal or greater than required strength, U = 1.2D + 1.6L and U= 1.4D
(2) ACI 318 design strength shown must be equal or greater than required strength, U = 1.4D + 1.7L
(3) Smaller width possible with deeper beam size. BSF unit must be located near the top of the beam section for minimum beam height shown.

This BSF manual presents the concept, engineering and design examples that illustrates conformance to Standard Industry Practice. By specifying a BSF connection the user assured of connection components of known load capacity designed to meet AISC and ACI 318 strength requirements. Also, full-scale load tests have verified the engineering design methodology. The BSF connection system can be easily incorporated into typical building designs.

PART 2 - ENGINEERING

SOURCE AND PURPOSE

To provide an independent evaluation of the BSF system, JVI contracted the services of Raths, Raths & Johnson, Inc., Structural Engineers of Willowbrook, Illinois, to provide an engineering evaluation program of the Norwegian designed and produced BSF connections for conformance to design, material and fabrication standards for construction in the United States. The engineering was performed by Robert W. Kritzler, S.E., P.E., under the direction of Donald C. Raths, S.E., Principal. BSF connection fabrication drawings and design examples show dimensional, material, and fabrication requirements for use of the BSF connections in the United States. The fabrication drawings are provided in English units for production in the United States, and in metric units for production in Norway. The BSF beam/column reinforcing design examples show typical conceptual solutions developed for utilizing the BSF connections in precast concrete beams and columns.

DESIGNERS ROLE AND RESPONSIBILITY

For any Building Project, it is the responsibility of the Engineer-of-Record (EOR) to establish all the project design criteria, determine all applicable Code design loads, prepare plans and specifications, and undertake all necessary and required designs including the primary structural concrete components and all permanent connections for gravity and lateral loads.

The BSF connection system is a proprietary structural component that the Engineer-of-Record (EOR) specifies for a concrete beam end connection. The information provided in this technical reference for the use of the BSF connection system is not a substitute for the Engineer's judgement and skill in creating a building with appropriate structural stability and permanent connections capable of withstanding all required gravity, lateral and torsional forces. The example framing details presented in this reference are concepts intended only as a guide for the Design Engineer's (EOR) consideration, and are not suitable for use "as is" for building construction. Neither JVI nor any of its consultants or suppliers have any Design Engineer (EOR) responsibility, or responsibility for Contractor use or application of the BSF connection system.

TORSION

The BSF connections are intended to be a vertical gravity connection. Torsional forces imposed on the beam must be resisted by either (1) designing the beam to carry torsion and providing permanent torsion connections between the beam and support columns; or (2) by permanent moment connections between the deck members (hollow core slab or double tee) and the support beam to eliminate beam torsion. Option (2) requires consideration of volume change forces in designing the deck-to-beam connections.

Multiple BSF connections in beam ends may be used to address torsion. However, beam torsional rigidity must be considered relating to "four-point-support" load distribution, and the need for special erection shimming adjustments.

DESIGN CONCEPT

The BSF connectors are composed of two steel boxes, one cast in the column (Column Box Unit) and the other, into which a steel slide (knife plate) is placed, in the beam end (Beam Box Unit). After positioning the beam into its final position, the slide is shifted from the steel box in the beam end into a slot in the steel column box. Figure 1 shows the principle of this connection.

The sliding knife plate of the BSF connection cantilevers past the beam end to transfer vertical load to the bottom of the column box in bearing. Refer to Fig. 1. This produces a completely hidden, hinged connection that allows small axial movements in the joint. The vertical force transfer between the steel beam box unit and the cantilevered knife plate is through bearing reactions at Points A and B in Fig. 1(e).

The force transfer between the BSF beam box unit and the beam concrete can be represented by an internal truss calculation model, shown in Fig. 2. The truss model, also generally referred to as a strut-and-tie model, consists of concrete compression struts (on both sides of beam box) and reinforcement acting as tension ties and struts. This internal truss analogy for the end of the concrete beam is a design methodology which satisfies equilibrium, and can be used to proportion the concrete and reinforcement to satisfy strength requirements. Internal load paths are defined to account for all forces acting on the end region of the beam, including BSF connection vertical and axial internal forces, concentrated loads, prestressing force, and geometric effects, such as concrete openings and voids.

The BSF connection allows for beam axial volume change movements due to shrinkage, creep and temperature change by sliding of the knife plate within the column box and beam box. Sliding friction in the column box induces an axial tensile force on the knife plate and beam box, shown in Fig. 2, which is transferred to the concrete beam by longitudinal reinforcement welded to the bottom of the beam box.

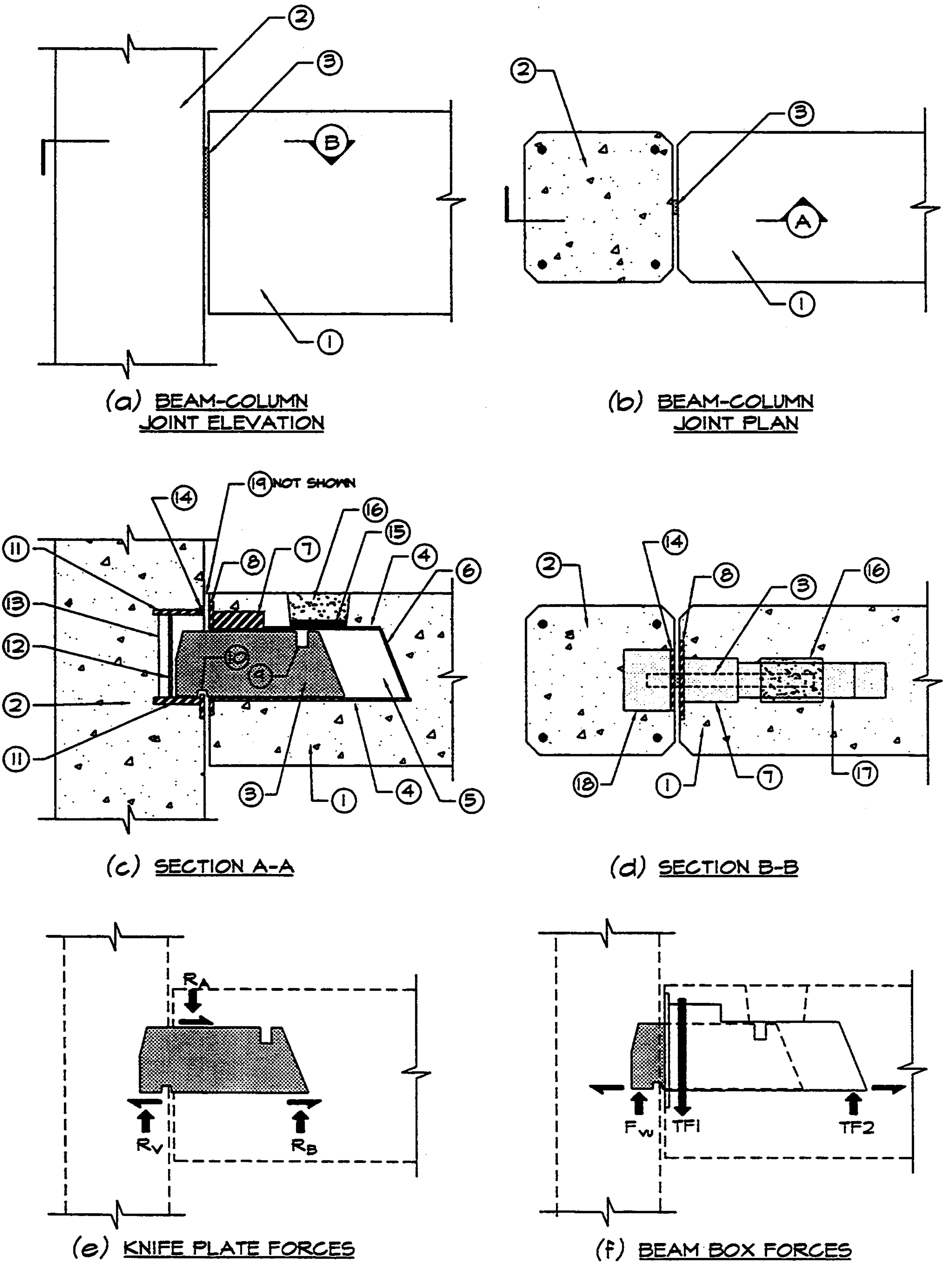
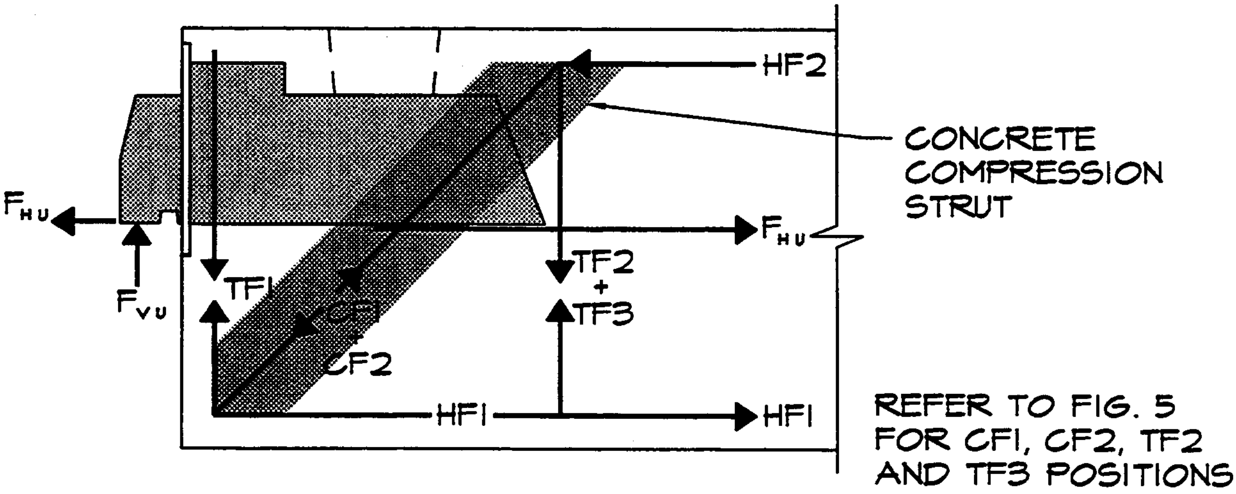


Fig. 1 The BSF Connector System

1. PRECAST BEAM
2. PRECAST COLUMN
3. STEEL KNIFE PLATE
4. STEEL BEAM BOX TOP AND BOTTOM PLATES
5. STEEL BEAM BOX SIDE PLATE(S)
6. STEEL BEAM BOX END PLATE
7. STEEL BEAM BOX TOP HALF-ROUND SADDLE FOR DIRECT-CONTACT SUPPORT OF BEAM-END SEMI-CIRCULAR-BENT HANGER REINFORCEMENT
8. STEEL BEAM BOX FRONT PLATE WITH SLOT FOR KNIFE PLATE
9. NOTCH IN KNIFE PLATE AT BEAM POCKET TO PERMIT CROW-BAR SLIDING PLACEMENT OF KNIFE PLATE INTO COLUMN BOX. OPTIONALLY, A LOOSE AND TEMPORARY LUG PLATE ("LOCK PIN") IS LEFT IN PLACE IN THE NOTCH TO ASSIST CROW BAR PLACEMENT
10. NOTCH IN KNIFE PLATE, AT COLUMN BOX PLATE, TO MECHANICALLY LOCK KNIFE PLATE INTO COLUMN BOX
11. STEEL COLUMN BOX TOP AND BOTTOM PLATES
12. STEEL COLUMN BOX END PLATE
13. STEEL COLUMN BOX SIDE PLATE(S)
14. STEEL COLUMN BOX FRONT PLATE WITH SLOT FOR KNIFE PLATE
15. BEAM POCKET BOTTOM SEAL
16. GROUT IN BEAM POCKET. PURPOSE OF BEAM POCKET IS TO PROVIDE ERECTION ACCESS TO KNIFE PLATE
17. STEEL BEAM BOX ASSEMBLY
18. STEEL COLUMN BOX ASSEMBLY
19. FIREPROOFING GROUT AND JOINT SEALANT

Fig. 1 The BSF Connector System



F_{VU}	=	Vertical support load
F_{HU}	=	Horizontal support load due to sliding friction restraining axial volume change
TF1	=	Vertical equilibrium tension force on beam box, from Fig. 1(f) ("Hanger Rebar Steel")
TF2	=	Vertical equilibrium tension force resulting from concrete compression below beam box, from Fig. 1(f)
TF3	=	Tension force equal to TF1 - TF2 (Equals F_{VU})
CF1	=	Compression force resulting from the inclined component of TF2 (Also refer to Fig. 5)
CF2	=	Compression force resulting from the inclined component of TF3 = TF1 - TF2 (Also refer to Fig. 5)
HF1	=	Horizontal tension force component of the inclined compression strut, at beam bottom
HF2	=	Horizontal compression force component of the inclined compression strut, at beam top

Fig 2. Simplified Truss Model

The truss analogy is a classic concept for understanding concrete structural behavior, and has traditionally been used to determine the ultimate strength of beam shear reinforcement (stirrups) acting as a series of web tension members with the concrete acting as web diagonal compression struts. The flexural concrete compression zone and the flexural reinforcement are the top and bottom chords of the analogous truss. The truss analogy has also been used in reinforced concrete design for deep beams, dapped-ends of beams, and brackets and corbels.

Although not included in the internal truss model, normal shear reinforcement (stirrups) is used in the beam around the BSF connection, distributed along the beam at normal spacing in the beam end. The beam stirrups provide confinement of concrete against lateral failure and improved bond of flexural reinforcement, as well as additional beam shear capacity and control of elastic crack widths at ultimate load.

COMPLIANCE WITH USA DESIGN AND CONSTRUCTION STANDARDS - AISC AND ACI

The BSF connection steel components: the column box unit, the beam box unit and knife plate, were designed in Norway in accordance with Norwegian Standard NS3472E, "Steel Structures Design Rules", 2nd edition, April 1985. The materials and fabrication for BSF units manufactured in Norway meet the requirements of this standard, as well as referenced Norwegian standards for workmanship and tolerances. The Norwegian design standard utilizes limit states design principles similar to the AISC LRFD Design Specification.

The designs of the five standard BSF connections have been checked using the AISC "Load and Resistance Factor Design Specification for Structural Steel Buildings", (LRFD) December, 1993. In addition, a sixth, larger capacity BSF unit has been designed in Norway and will also be checked using this AISC LRFD Specification. The BSF unit fabrication drawings included in this technical reference incorporate design changes required to meet or exceed AISC design standards for the design (ultimate) strengths listed.

The structural steel grades used in Norway are DIN Standards St 52-3 and St 44-2, which are generally equivalent to ASTM A572, Grades 50 and 42, respectively. The welding electrodes conform to the "Specification for Carbon Steel Filler Metals for Gas Shielded Arc Welding", AWS A5.18-79, of the American Welding Society.

Fabrication of the BSF connection units used in the USA also meets or exceeds the applicable provisions of the American Welding Society "Structural Welding Code - Steel", AWS D1.1, including welding technique, qualification, workmanship and inspection.

The BSF unit fabrication drawings included in this reference specify the design, material, and fabrication requirements, and design strength, for production of the units in the United States, or in Norway and exported for building construction in the United States. The design strength is the strength limit state capacity calculated in accordance with the AISC LRFD Design Specification. This connection design strength shall equal or exceed the required strength based on subjecting the structure to all appropriate factored load combinations.

The transfer of forces from the BSF column box unit and the BSF beam box unit to the concrete column and beam, respectively, must be designed by the Engineer-of-Record to satisfy the requirements of ACI 318-89 (Revised 1992) "Building Code Requirements for Reinforced Concrete."

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).

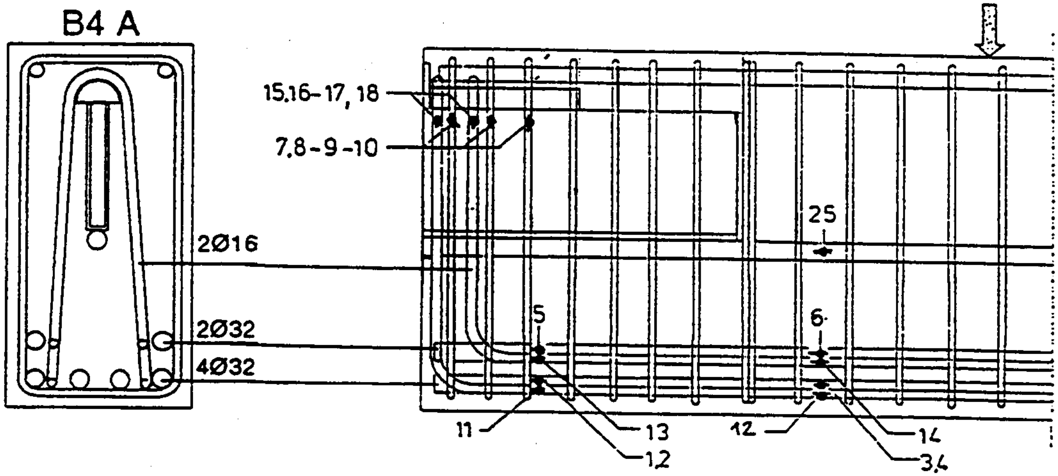
Table 1 - BSF Test Results - Partial Summary

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

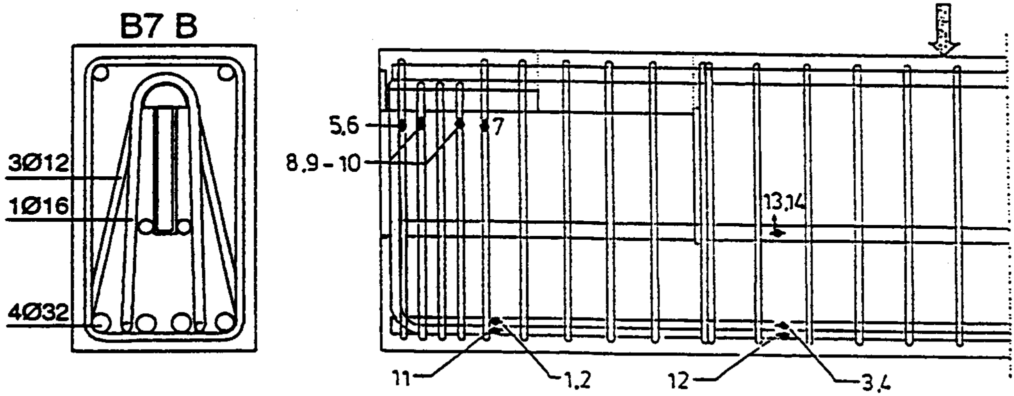
SINTEF Test	Type of Failure
B4A	Steel box failure, test stopped; half-round weld failure at 90 kips
B5A	Concrete shear - tension
B5B	Concrete shear / bond to bottom of BSF beam box
B6A	Push-out of concrete; half-round weld failure at 67 kips; main bot. reinforcement bond failure
B7B	Push-out of concrete;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



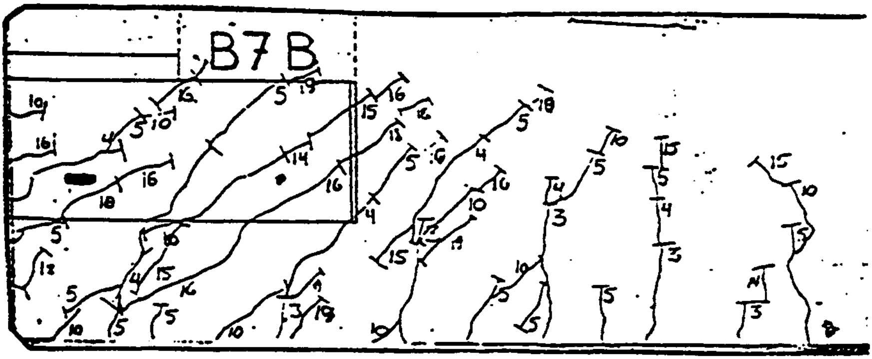
Location of Reinforcement and Strain Gages

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

BEAM/COLUMN REINFORCEMENT DESIGN EXAMPLE

A design example for the BSF beam box and column box anchoring reinforcement is shown below for a BSF 200/50 unit. This example is for the full capacity of the 200/50 unit.

Given:

The beam with BSF 200/50 connection shown on drawing R200/50, page 2-20.

F_{VD} = 69.7 kips (service dead load)
F_{VL} = 46.1 kips (service live load)
f_Y for all reinforcement = 60 ksi
f'_C = 5000 psi, normal weight concrete

Required Strength:

F_{VU} (AISC) = 1.2D + 1.6L = 1.2 (69.7) + 1.6 (46.1) = 157 kips

AISC Design Strength, ϕRn = 157 kips $\geq F_{VU}$

BSF 200/50 unit capacity adequate

F_{VU} (ACI 318) = 1.4D + 1.7L = 1.4 (69.7) + 1.7 (46.1) = 176 kips
F_{HU} (ACI 318) = 1.4T = 1.4 μ_S (F_{VD}) = 1.4 (0.4) (69.7) = 39.0 kips

The design of BSF hardware and anchoring reinforcement, and selection of the critical load location (shown below), results in a ductile connection not subject to brittle failures from minor variations in materials and reinforcing location. Therefore, the additional load factor of 1.3 as defined in PCI Handbook Section 6.3 is not used.

Beam Truss Analogy Forces:

The internal truss analogy force transfer between the BSF box unit and beam concrete is shown in Fig. 5.

Distance from face/beam to force F_{VU}
a = Joint width + knife notch width + 0.75 x knife nose bearing length
a = 0.6 + 1.2 + 0.75 x 2.56 = 3.72 in.

From Table 3, use X = 2.2 inches and U = 5 inches
Bottom length of beam box = 23 ⅜ inches (drawing S200/50)

Y = 23⅜ - X - 0.5U = 23.4 - 2.2 - 0.5 x 5 = 18.7 in. - use 18 in. (conservative for TF1 tension force and TF2 concrete bearing length)

$$TF1 = F_{VU}\frac{(a + X + Y)}{Y} = 176\frac{(3.72 + 2.2 + 18)}{18} = 234 \text{ kips}$$

$$TF2 = TF1 - F_{VU} = 234 - 176 = 58 \text{ kips}$$

$$TF3 = TF1 - TF2 = F_{VU} = 176 \text{ kips}$$

Beam Truss Analogy Forces - Continued

Try Z = 5 in. to center of TF3

$$Z_R = \frac{58(0) + 176(5)}{58 + 176} = 3.8 \text{ in.}$$

Try 6 sets of closed stirrups at 2 in. spacing for TF2 and TF3

$$\text{TF2/TF3 zone width} = 6\ (2) = 12 \text{ in.} \leq \frac{h}{2.5} = \frac{36}{2.5} = 14.4 \text{ in. ok.}$$

TF2/TF3 zone extends to left of TF2 by 2.2 in. $\approx \frac{U}{2}$ $\therefore$ zone width/location ok.

$$\text{Length CF1} = \sqrt{(18)^2 + (28)^2} = 33.3 \text{ in.}$$

$$\text{Length CF2} = \sqrt{(18 + 5)^2 + 28^2} = 36.2 \text{ in.}$$

CF = CF1 + CF2 = TF2 (33.3/28) + TF3 (36.2/28) = 297 kips

HF1 = HF2 = TF2 (18/28) + TF3 (23/28) = 182 kips

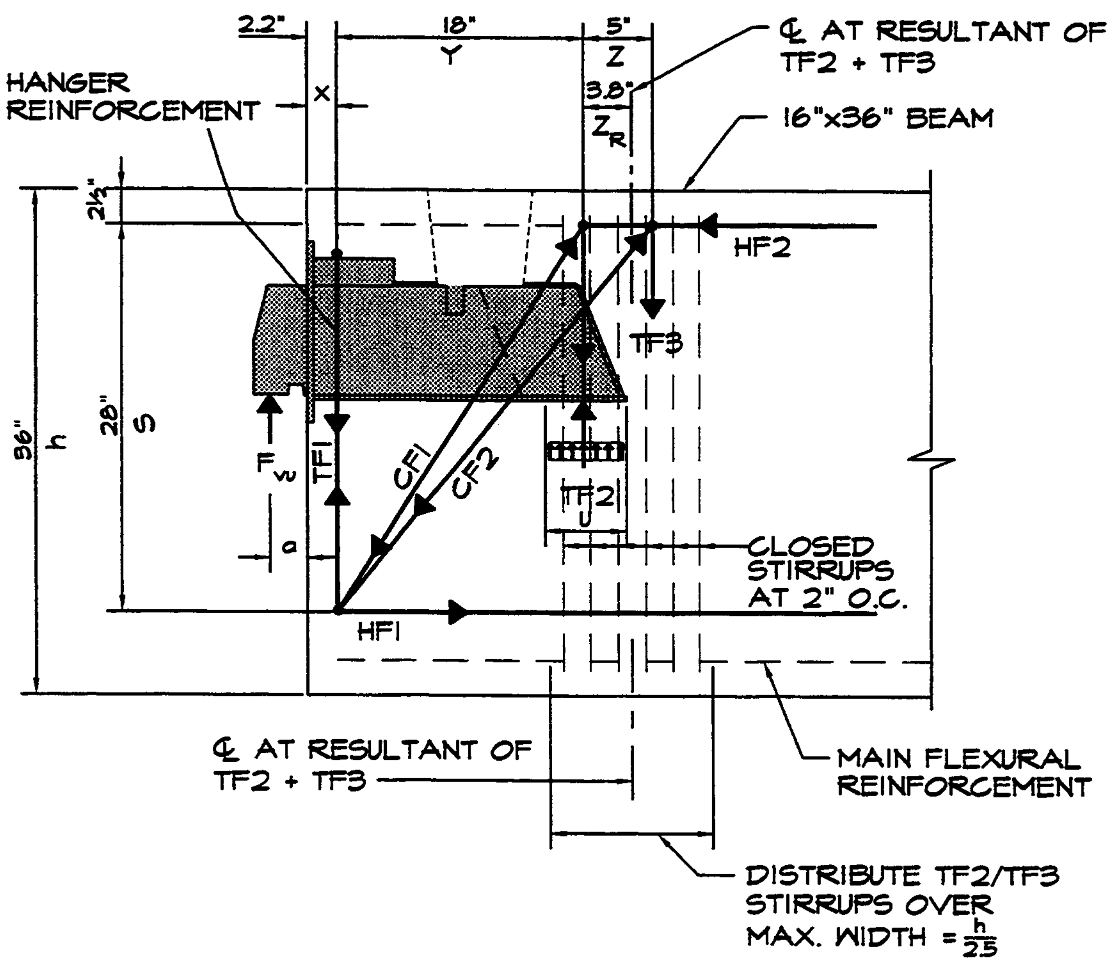


Fig. 5 Example Beam End Truss Analogy for BSF 200/50 Connection

Vertical Shear Along Diagonal Strut:

To limit diagonal concrete stresses and prevent crushing of the concrete, ACI 318-89 Section 11.5.6.8 limits shear strength to a conservative value of $V_S = 8\sqrt{f'_C}$ psi, regardless of the amount of shear reinforcement. Thus from ACI 318 equations 11-1, 11-2 and 11-3, the maximum nominal concrete vertical shear strength in a beam is $V_S + V_C$:

$$v_U \leq \phi 10 \sqrt{f'_C} \text{ psi} = .85\ (10)\ \sqrt{5000} = 601 \text{ psi}$$

Vertical shear area at concrete strut CF,

A_V = (beam width - BSF box width) d
= (16 - 2.69)(28 + 2.5) = 406 square inches

v_U = TF1/406 = 234 (1000)/406 = 576 psi < 601

Concrete shear capacity adequate at compression strut.

Diagonal Compression Strut:

Inclined width of strut CF based on spacing of closed stirrup reinforcement carrying tension forces TF2 and TF3. For 6 sets of stirrups at 2 inch spacing, shown in Fig. 6:

$$\text{Strut CF angle } \theta = \arctan\left(\frac{28}{22}\right) = 51.8 \text{ degrees}$$

$$t = 12\ (\sin\theta) = 9.4 \text{ in.}$$

Strut horizontal width, w = beam width - BSF box width
= 16 - 2.69
= 13.3 in.

$$f_{CU} = \frac{CF}{w(t)} = \frac{297\ (1000)}{9.4\ (13.3)} = 2376 \text{ psi}$$

Recommended values for effective compressive strength, f_{CE}, are given by MacGregor [1] for deep beam truss models. The ACI Code does not address this problem except by reference in the Commentary. For compression struts, $f_{CE} = 0.5 f'_C$ is recommended. The Norwegian Concrete Design Standard limits f_{CE} to approximately $0.55 f'_C$ and the AASHTO Guide Specification for Design of Segmental Concrete Bridges limits compression strut f_{CE} to $0.51 f'_C$.

f_{CE} = $0.5 f'_C = 0.5\ (5000)$
= 2500 psi > f_{CU} = 2376

Diagonal concrete compression strut adequate

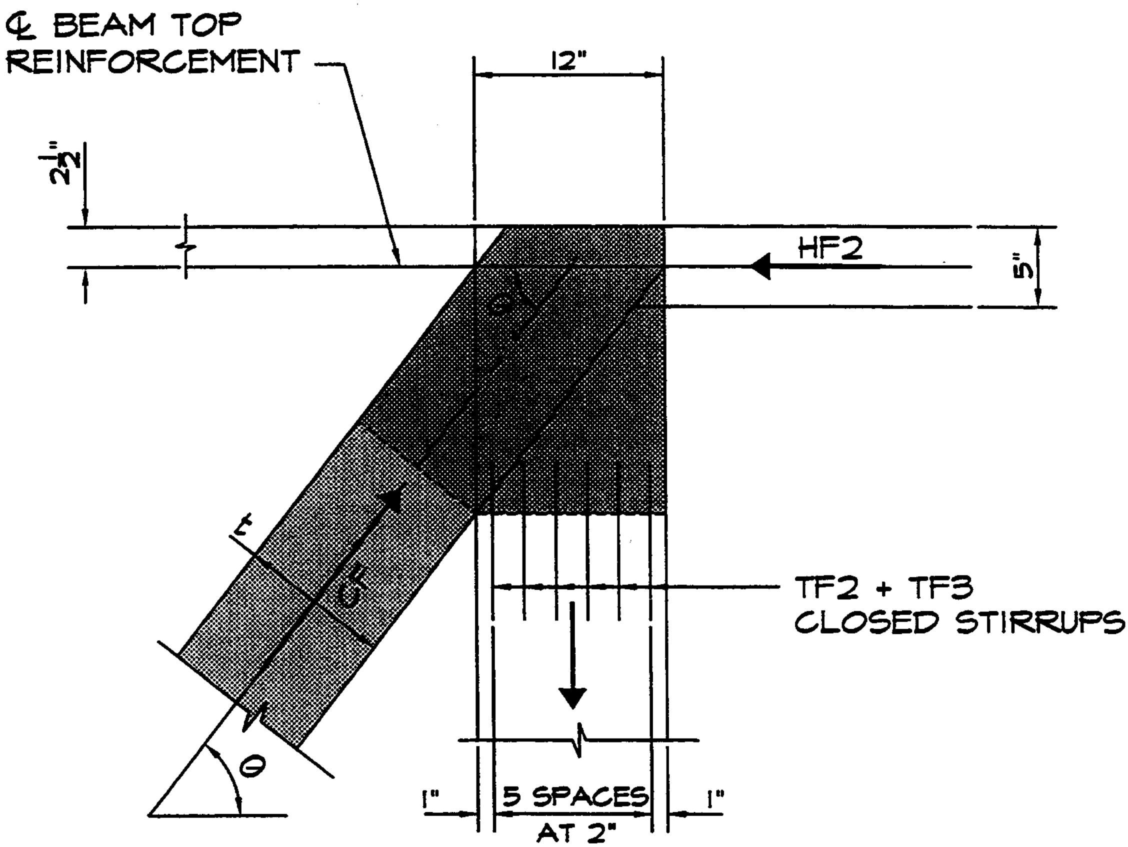


Fig. 6 Diagonal Compression Strut at BSF 200/50 Connection

Required Reinforcement For TF1 (Hanger Reinforcement):

$$A_{S1} = \frac{TF1}{\phi_T(f_Y)} = \frac{234}{0.9(60)} = 4.33 \text{ sq. in.}$$

Use 2 - #7 welded to beam box front plate
Use 4 - #7 and 2 - #6 bent to fit in direct contact over BSF box front end top saddle

NOTE: ASTM A706 deformed bars required for welding to BSF front plate and for small diameter bending to fit BSF top saddle

$$A_{S1} = 6\,(0.60) + 2\,(0.44) = 4.48 \text{ sq. in.}$$

Check C.G. of reinforcement from front face of BSF unit:

$$x = [2(0.60)(0.5 + \frac{0.875}{2}) + 2(0.60)(1.5) + 2(0.60)(3) + (2)(0.44)(5)] / 4.48$$

$$= 2.4 \text{ in.} \approx 2.2 \text{ ok.}$$

Required Reinforcement For TF2:

$$A_{S2\text{-}1} = \frac{TF2}{\phi_T(f_Y)} = \frac{58}{0.9(60)} = 1.07 \text{ sq. in.}$$

Use 1½ sets #4 double stirrups = 1.2 sq. in.

Required Reinforcement For TF3:

$$A_{S2\text{-}2} = \frac{TF3}{\phi_T(f_Y)} = \frac{176}{0.9(60)} = 3.26 \text{ sq. in.}$$

Use 4½ sets - #4 double stirrups = 3.6 sq. in.

Required Reinforcement For HF1:

$$A_{S5} = \frac{HF1}{\phi_B(f_Y)} = \frac{182}{0.9(60)} = 3.37 \text{ sq. in.}$$

From TF1 reinforcement, A_{S1} = 4.48 sq. in. Therefore no additional reinforcement required. Extend TF1 horizontal reinforcement to be fully developed in tension on beam interior side (towards mid-span) of TF3.

Compression Force HF2 in Top of Beam:

HF2 = 182 kips
2 - #8 and 2 - #6 beam top reinforcement, A_{S3}

Compression Capacity = $\phi_B\, f_Y\, A_S$ (TF2/TF3 zone width/ℓ_d compr.)* + f_{CE} (A_C-A_{S3})
* ratio = 1.0 max.

A_S = A_{S3} eff = 2(0.79)(12/18) + 2 (.44)(12/14) = 1.81 sq. in.
A_C = 2.5 (2)(b) = 2.5 (2)(16) = 80.0 sq. in.

Compression Capacity = 0.9 (60)(1.81) + .5 (5)(80.0 - 2.46) = 98 + 194
= 292 kips > 182

Use 2 - #8 and 2 - #6 beam top reinforcement. Reinforcement to be fully developed in compression between beam end and TF3.

Required Reinforcement For F_{HU}:

$$A_{S4} = \frac{F_{HU}}{\phi_T(f_Y)} = \frac{39.0}{0.9(60)} = 0.72 \text{ sq. in.}$$

Use 1 - #9 = 1.00 sq. in. welded to bottom of beam box rather than 1 - #8 in consideration of beam box displacements resulting from concrete cracking at ultimate loading.

Splitting Forces at Bottom 90° Bend of TF1/HF1:

Partek Ostspenn engineers in Norway and the Norwegian Standard NS 3473E "Concrete Structures Design Rules" Section 12.9.5 require providing transverse reinforcement at the bend, or use of substantially larger bend diameters at the TF1/HF1 bend.

For standard minimum ACI 318 90° bend diameters, provide $A_{S6} = 0.4\ A_{S1}$ for each side of beam, using transverse reinforcement (A_{S6}) for the portion of A_{S1} on each side of beam centerline:

$$A_{S6} = 0.4\,(4.48)(0.5) = 0.9 \text{ sq. in.}$$

Use 3 - #3 transverse ties and 2 - #4 longitudinal hairpins,

$$A_{S6} = 3\,(2)(0.11) + 2\,(0.2) = 1.06 \text{ sq. in.}$$

Column Box Forces:

The forces acting on the column box are shown in Fig. 7.

$$F_{VU} = 176 \text{ kips}$$

Column box design based on distributing 20 percent of bearing load to column box top horizontal plate.

$$F_{VU}(\text{top}) = 176\,(0.2) = 35.2 \text{ kips}$$

$$F_{VU}\ (\text{bottom}) = 176\,(0.8) = 141 \text{ kips}$$

$$N_U = F_{HU} = 39.0 \text{ kips}$$

Vertical load eccentricity, $e = 5.31\,(0.5) - 2.56\,(1-0.75) = 2.01$ in.

$$S = \frac{F_{VU}(e)}{10.75} = \frac{176(2.01)}{10.75} = 32.9 \text{ kips}$$

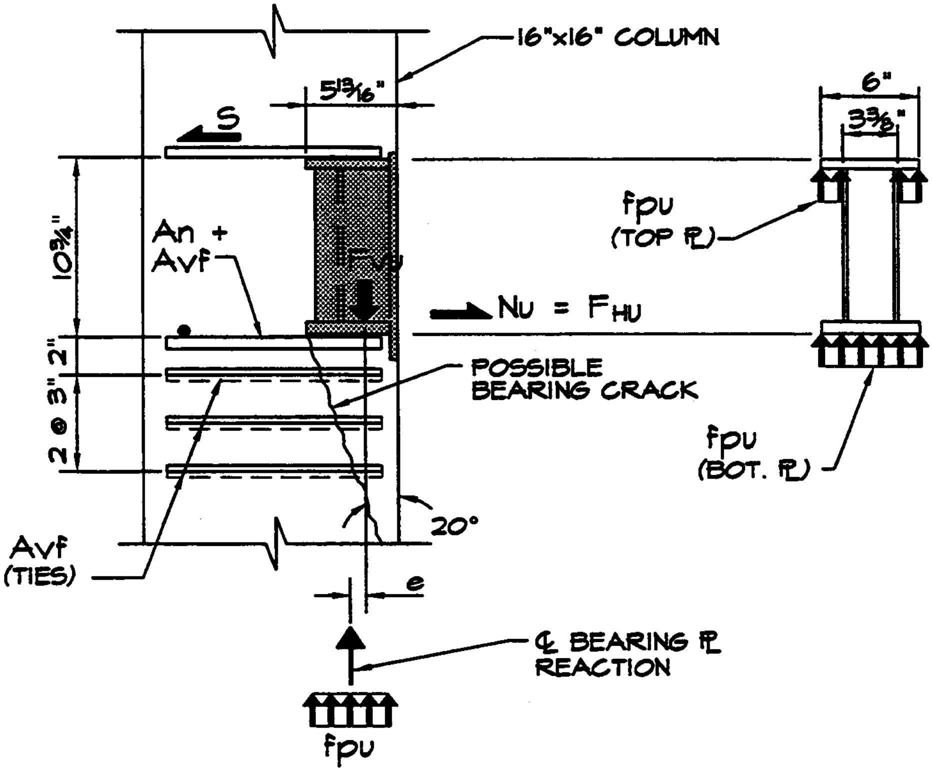


Fig. 7 Example Column Reinforcement at BSF 200/50 Connection

Column Box Top Horizontal Anchorage:

$$A_S = \frac{S}{\phi_T(f_Y)} = \frac{32.9}{0.9(60)} = 0.61 \text{ sq. in.}$$

Use 2 - #5 welded to top plate = 0.62 sq. in.

Bearing Reinforcement Under Column Box:

Use PCI Design Handbook methodogy for reinforced concrete bearing, 3rd Edition Section 6.9 using shear-friction theory. Refer to Fig. 7.

$$A_N = \frac{N_U}{\phi_V(f_Y)} = \frac{39.0}{0.85(60)} = 0.76 \text{ sq. in.}$$

Bearing Reinforcement Under Column Box - Continued

A_{CR} = Area of inclined crack plane + triangular vertical plane on each side of 6 in. wide bearing area

A_{CR} = 6 (5.81/sin 20°) + 2 (5.81)(5.81/tan 20°)(0.5) = 195 sq. in.

$$\mu_E = \frac{1000\ A_{CR}\mu}{F_{VU}} = \frac{1000(195)(1.4)}{141{,}000} = 1.9 < 3.4$$

$$A_{VF} = \frac{F_{VU}(\text{bot.})}{\phi_V(f_Y)\mu_E} = \frac{141}{0.85(60)(1.9)} = 1.46 \text{ sq. in.}$$

Use 2 - #6 welded to bottom plate

Use 3 - sets #3 double ties below bottom plate

A_{VF} = 2 (0.44) - 0.76 + 3(4)(0.11) = 1.44 sq. in. ≈ 1.46

GENERAL DESIGN REQUIREMENTS

Table 3 - BSF Beam Reinforcing Location Requirements:

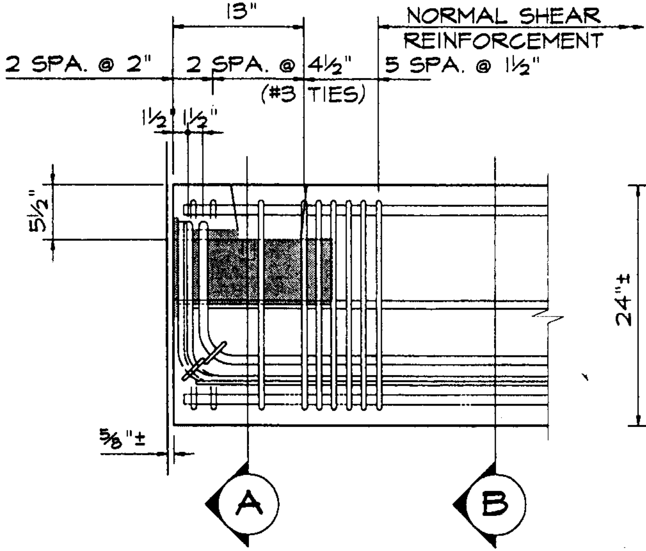
BSF Unit	Fig. 5 - Max. Dim. X (in.)	Fig. 5 - Dim. U (in.)
150/20	2.4	4
200/20	2.6	5
200/30	2.6	5
200/40	2.7	5
200/50	2.5	5
250/50	3.3	5

X = location of TF1 hanger rebar steel
U = length of concrete bearing block under interior end of beam box

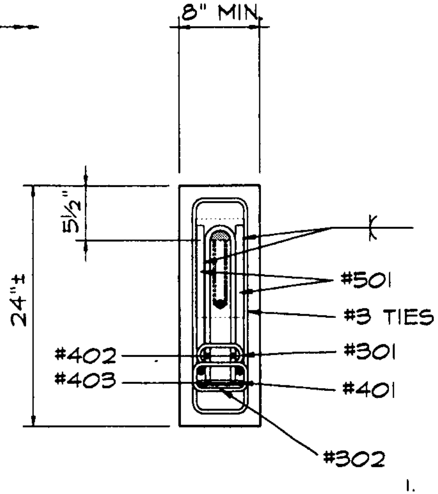
REFERENCES

[1] James G. MacGregor. Reinforced Concrete Mechanics and Design, 2nd Edition, Chapter 17, Prentice-Hall, Inc., Englewood Cliffs, New Jersey, 1988.

- **BEAM/COLUMN REINFORCEMENT DESIGN EXAMPLES**



BEAM END ELEVATION



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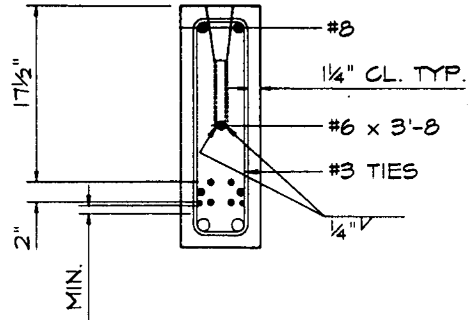


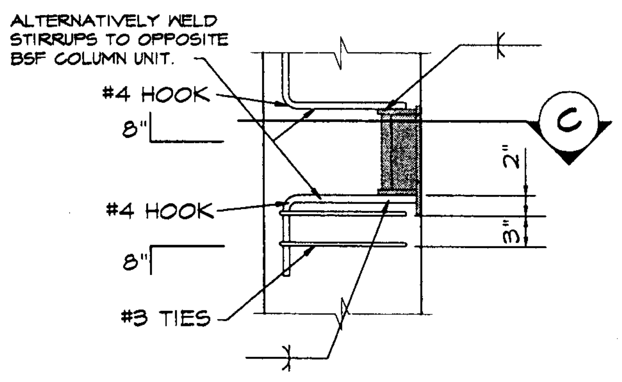
SECTION "B"

DESIGN EXAMPLE - NOT FOR CONSTRUCTION

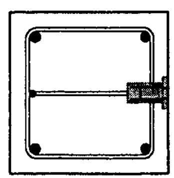
GENERAL NOTES

1. All concrete work to be in accordance with ACI 318-89 "Building Code Requirements for Reinforced Concrete", the PCI Design Handbook, 4th Edition, and PCI MNL-116-85 "Manual for Quality Control for Plants and Production of Precast and Prestressed Concrete Products.
2. Concrete to be normal weight (150 pcf) and to have a minimum 28-day compressive strength of $f'c = 5000$ psi.
3. All welded reinforcing bars and "hanger bars" (#402 & #403) to be ASTM A706 Low-Alloy Steel Deformed Bars (Grade 60). All other reinforcing bars to be ASTM A615-Grade 60.
4. Reinforcing bar welding to be in accordance with AWS D1.4-79 "Structural Welding Code - Reinforcing Steel".

ACI 318 LOAD CAPACITY:
Vertical Design (Ultimate) Strength, ϕRn = 50 Kips (222 kn)
$(1.4D + 1.7L \le 50)$



COLUMN ELEVATION



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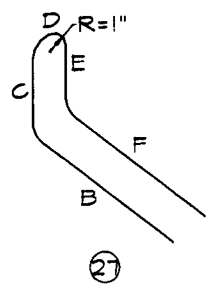

BAR BEND SCHEDULE								
MARK	TYPE	A	B	C	D	E	F	G
#301	T1	4"	4¾"	3"	4¾"	3"	NA	4"
#302	T1	4"	5½"	3½"	5½"	3½"	NA	4"
#401	17	NA	2'-2	4¾"	2'-2	NA	NA	NA
#402	27	NA	3'-0	1'-1	3"	1'-1	3'-0	NA
#403	27	NA	3'-0	1'-3	3"	1'-3	3'-0	NA
#501	17	NA	3'-7	1'-2	---	NA	NA	NA

BSF 150/20 REINFORCEMENT EXAMPLE
BSF CONNECTIONS
JVI / PARTEK ØSTSPENN A.S

JVI CONNECTIONS, INC.
7550 North Linder Avenue
Skokie, Illinois 60077 USA
Tel. 847/675-1560 - Fax 847/675-0083

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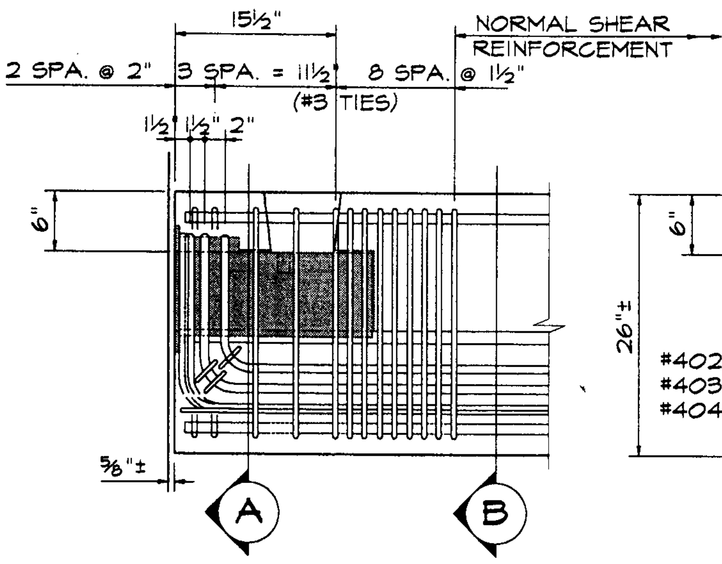
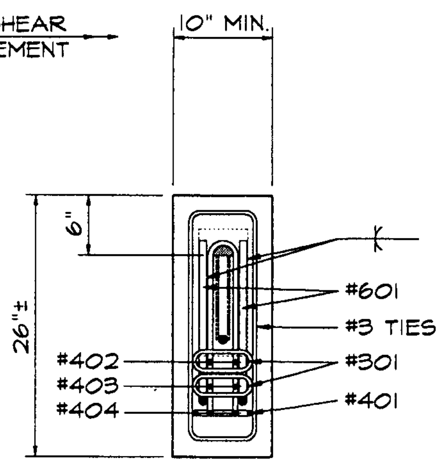


BEAM END ELEVATION



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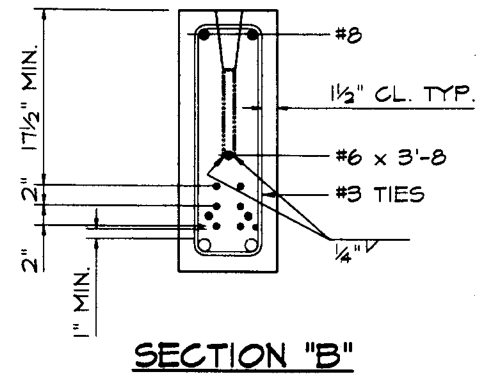

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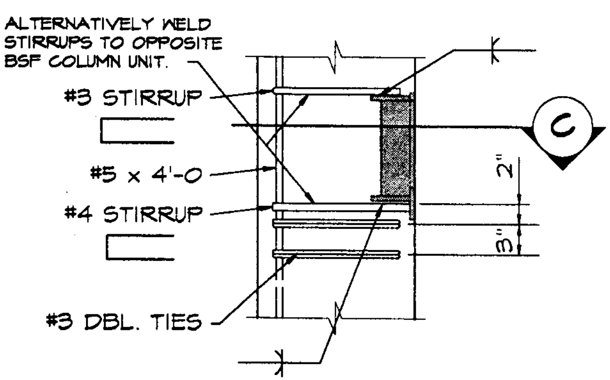
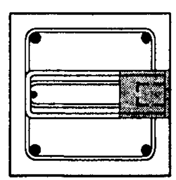
DESIGN EXAMPLE - NOT FOR CONSTRUCTION



COLUMN ELEVATION



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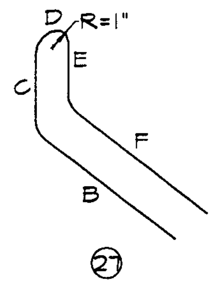

GENERAL NOTES

1. All concrete work to be in accordance with ACI 318-89 "Building Code Requirements for Reinforced Concrete", the PCI Design Handbook, 4th Edition, and PCI MNL-116-85 "Manual for Quality Control for Plants and Production of Precast and Prestressed Concrete Products.
2. Concrete to be normal weight (150 pcf) and to have a minimum 28-day compressive strength of f'c = 5000 psi.
3. All welded reinforcing bars and "hanger bars" (#402, #403 & #404) to be ASTM A706 Low-Alloy Steel Deformed Bars (Grade 60). All other reinforcing bars to be ASTM A615-Grade 60.
4. Reinforcing bar welding to be in accordance with AWS D1.4-79 "Structural Welding Code - Reinforcing Steel".

ACI 318 LOAD CAPACITY:
Vertical Design (Ultimate) Strength, ΦR_n = 75 Kips (334 kn)
$(1.4D + 1.7L \leq 75)$

BAR BEND SCHEDULE

MARK	TYPE	A	B	C	D	E	F	G
#301	T1	4"	6¼"	3"	6¼"	3"	NA	4"
#401	17	NA	2'-2	6"	2'-2	NA	NA	NA
#402	27	NA	2'-1	1'-0	3"	1'-0	2'-1	NA
#403	27	NA	3'-6	1'-2	3"	1'-2	3'-6	NA
#404	27	NA	3'-6	1'-4	3"	1'-4	3'-6	NA
#601	17	NA	4'-3	1'-3	---	NA	NA	NA

BSF 200/20 REINFORCEMENT EXAMPLE
BSF CONNECTIONS
JVI / PARTEK ØSTSPENN A.S

JVI CONNECTIONS, INC.
7550 North Linder Avenue
Skokie, Illinois 60077 USA
Tel. 847/675-1560 - Fax 847/675-0083

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SHEET NO. R200/20

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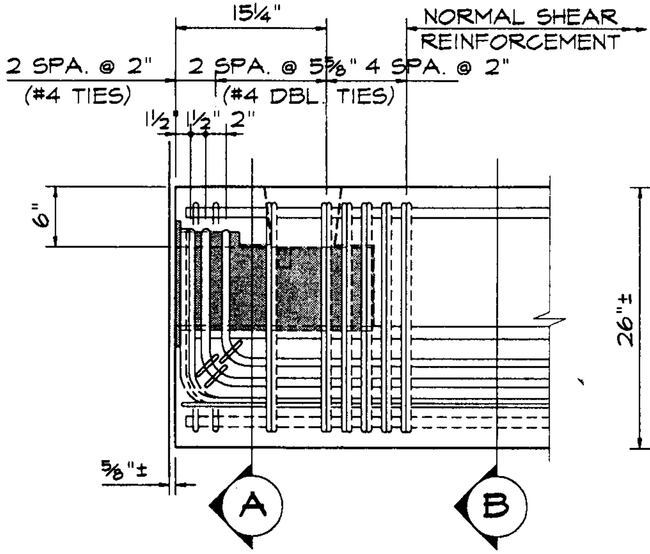
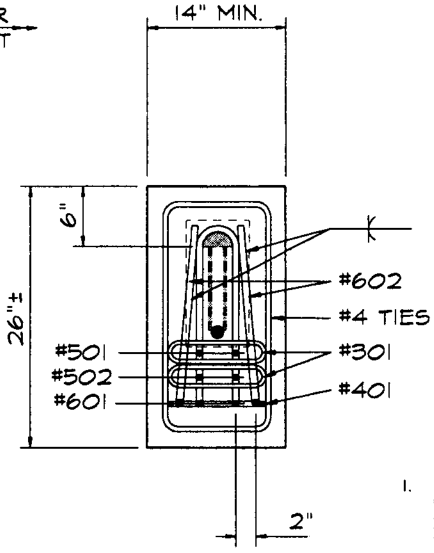


BEAM END ELEVATION



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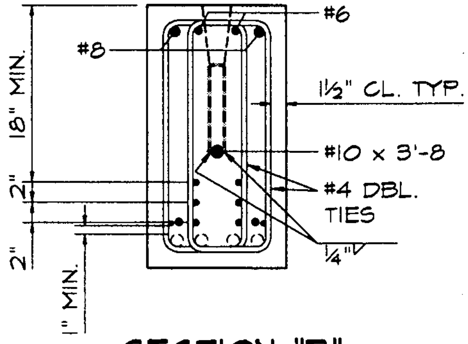


SECTION "B"

DESIGN EXAMPLE - NOT FOR CONSTRUCTION

GENERAL NOTES

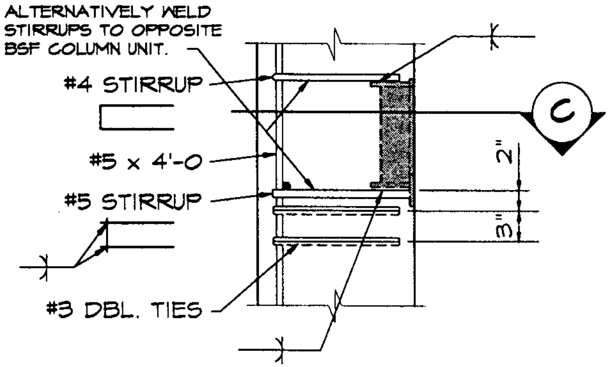
1. All concrete work to be in accordance with ACI 318-89 "Building Code Requirements for Reinforced Concrete", the PCI Design Handbook, 4th Edition, and PCI MNL-116-85 "Manual for Quality Control for Plants and Production of Precast and Prestressed Concrete Products.

2. Concrete to be normal weight (150 pcf) and to have a minimum 28-day compressive strength of f'c = 5000 psi.

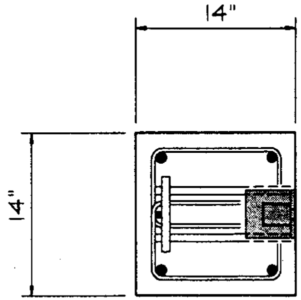
3. All welded reinforcing bars and "hanger bars" (#501, #502 & #601) to be ASTM A706 Low-Alloy Steel Deformed Bars (Grade 60). All other reinforcing bars to be ASTM A615-Grade 60.

4. Reinforcing bar welding to be in accordance with AWS D1.4-79 "Structural Welding Code - Reinforcing Steel".

ACI 318 LOAD CAPACITY:
Vertical Design (Ultimate) Strength, ΦRn = 110 Kips (489 kn)
(1.4D + 1.7L ≤ 110)



COLUMN ELEVATION



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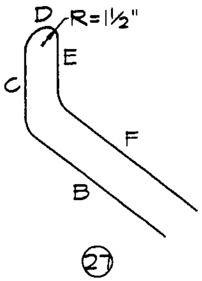

BAR BEND SCHEDULE								
MARK	TYPE	A	B	C	D	E	F	G
#301	T1	4"	10"	3"	10"	3"	NA	4"
#401	17	NA	2'-2	10"	2'-2	NA	NA	NA
#501	27	NA	3'-6	1'-1	4¼"	1'-1	3'-6	NA
#502	27	NA	3'-6	1'-3	4¼"	1'-3	3'-6	NA
#601	27	NA	4'-3	1'-5	4½"	1'-5	4'-3	NA
#602	17	NA	4'-3	1'-4	---	NA	NA	NA

BSF 200/30 REINFORCEMENT EXAMPLE
BSF CONNECTIONS
JVI / PARTEK ØSTSPENN A.S

JVI CONNECTIONS, INC.
7550 North Linder Avenue
Skokie, Illinois 60077 USA
Tel. 847/675-1560 - Fax 847/675-0083

DATE 2-9-96 | DRAWN SEA | CHECKED RWK | SCALE NTS

JOB NO.
SHEET NO. R200/30

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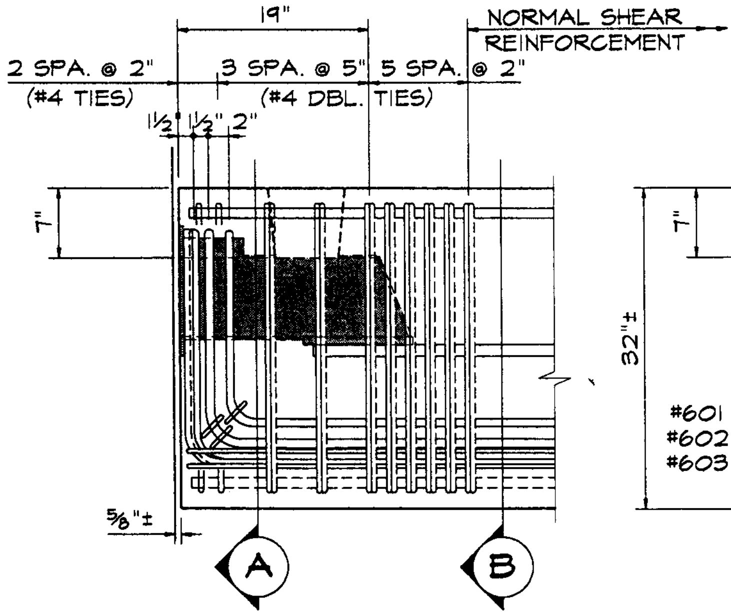
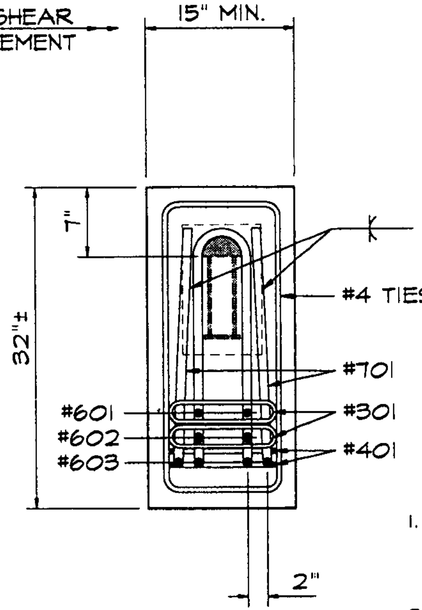


BEAM END ELEVATION



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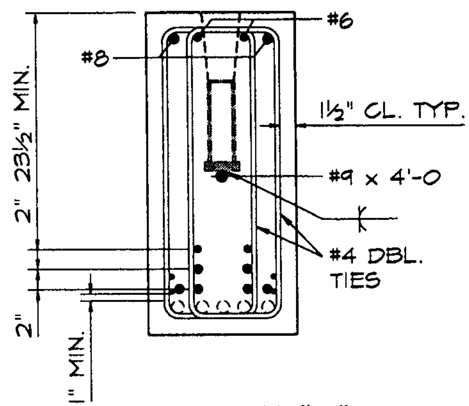

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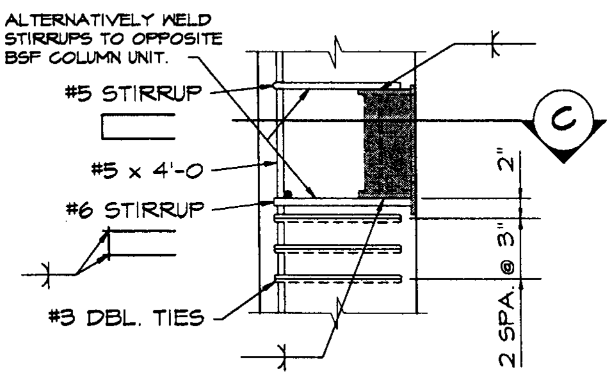
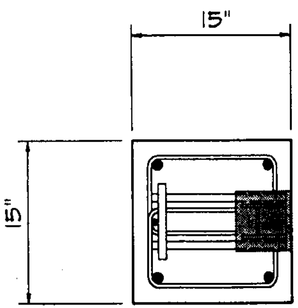


COLUMN ELEVATION



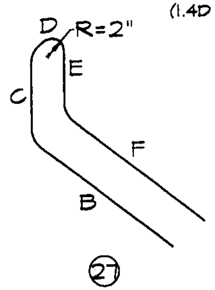
SECTION "C"

DESIGN EXAMPLE - NOT FOR CONSTRUCTION

GENERAL NOTES

1. All concrete work to be in accordance with ACI 318-89 "Building Code Requirements for Reinforced Concrete", the PCI Design Handbook, 4th Edition, and PCI MNL-116-85 "Manual for Quality Control for Plants and Production of Precast and Prestressed Concrete Products.
2. Concrete to be normal weight (150 pcf) and to have a minimum 28-day compressive strength of f'c = 5000 psi.
3. All welded reinforcing bars and "hanger bars" (#601, #602 & #603) to be ASTM A706 Low-Alloy Steel Deformed Bars (Grade 60). All other reinforcing bars to be ASTM A615-Grade 60.
4. Reinforcing bar welding to be in accordance with AWS D1.4-79 "Structural Welding Code - Reinforcing Steel".

ACI 318 LOAD CAPACITY:
Vertical Design (Ultimate) Strength, ϕR_n = 150 Kips (667 kn)
($1.4D + 1.7L \leq 150$)



BAR BEND SCHEDULE

MARK	TYPE	A	B	C	D	E	F	G
#301	T1	4"	1'-0	3"	11"	3"	NA	4"
#401	17	NA	2'-2	11"	2'-2	NA	NA	NA
#601	27	NA	4'-3	1'-6	5½"	1'-6	4'-3	NA
#602	27	NA	4'-3	1'-8	5½"	1'-8	4'-3	NA
#603	27	NA	4'-3	1'-10	5½"	1'-10	4'-3	NA
#701	17	NA	5'-0	1'-10	---	NA	NA	NA

BSF 200/40 REINFORCEMENT EXAMPLE
BSF CONNECTIONS
JVI / PARTEK ØSTSPENN A.S

JVI CONNECTIONS, INC.
7550 North Linder Avenue
Skokie, Illinois 60077 USA
Tel 847/675-1560 - Fax 847/675-0083

DATE	DRAWN	CHECKED	SCALE
2-9-96	SEA	RWK	NTS

JOB NO.
SHEET NO. R200/40

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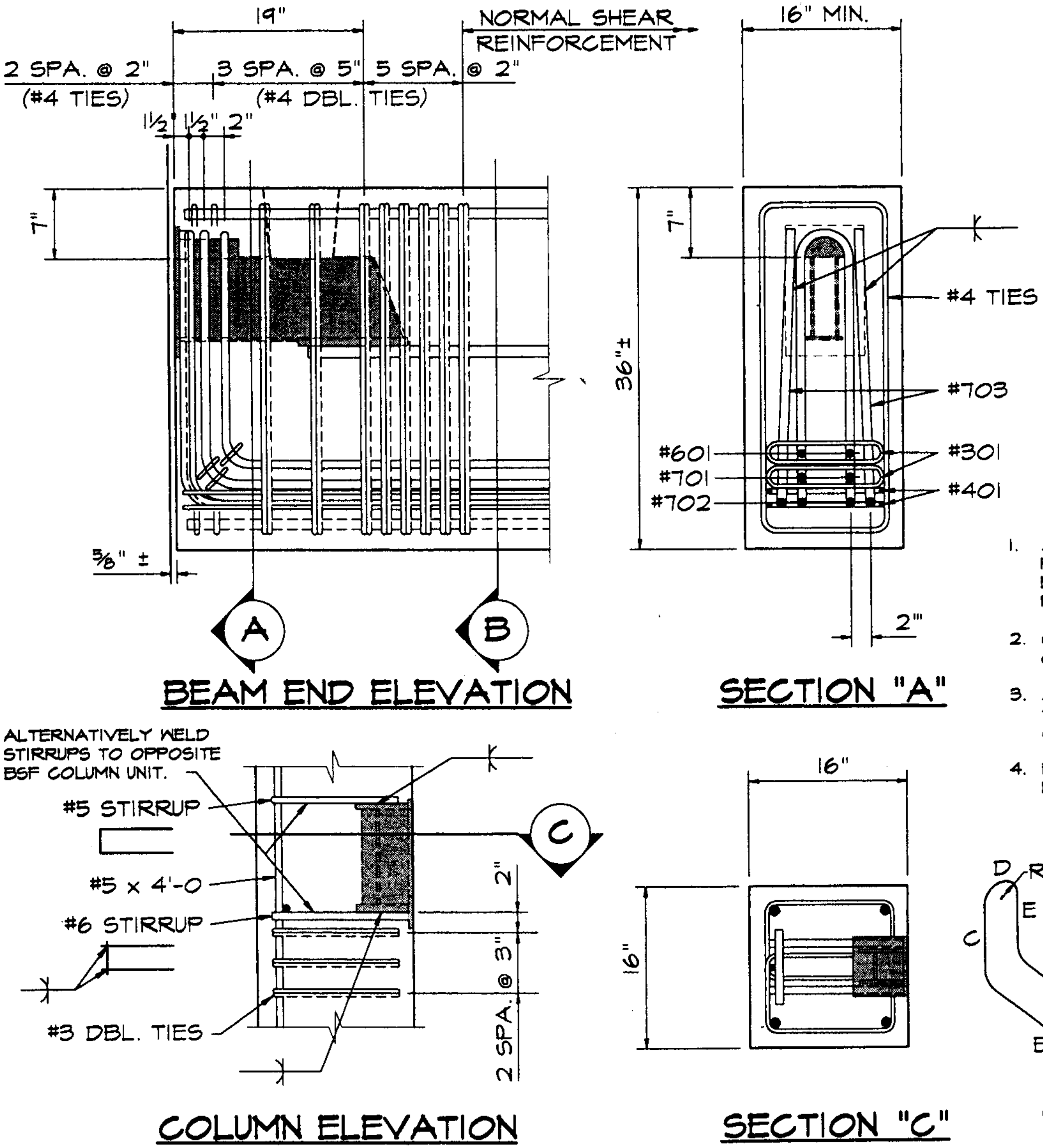
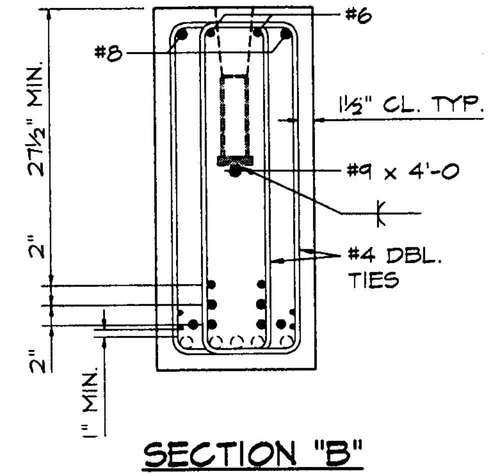


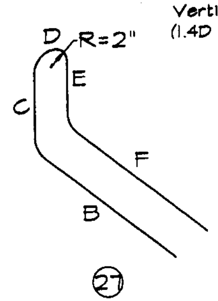


DESIGN EXAMPLE - NOT FOR CONSTRUCTION

GENERAL NOTES

1. All concrete work to be in accordance with ACI 318-89 "Building Code Requirements for Reinforced Concrete", the PCI Design Handbook, 4th Edition, and PCI MNL-116-85 "Manual for Quality Control for Plants and Production of Precast and Prestressed Concrete Products.
2. Concrete to be normal weight (150 pcf) and to have a minimum 28-day compressive strength of f'c = 5000 psi.
3. All welded reinforcing bars and "hanger bars" (#601, #701 & #702) to be ASTM A706 Low-Alloy Steel Deformed Bars (Grade 60). All other reinforcing bars to be ASTM A615-Grade 60.
4. Reinforcing bar welding to be in accordance with AWS D1.4-79 "Structural Welding Code - Reinforcing Steel".

ACI 318 LOAD CAPACITY:
Vertical Design (Ultimate) Strength, ϕRn = 175 Kips (778 kn)
$(1.4D + 1.7L \leq 175)$



BAR BEND SCHEDULE

MARK	TYPE	A	B	C	D	E	F	G
#301	T1	4"	1'-0	3"	1'-0	3"	NA	4"
#401	17	NA	2'-2	1'-0	2'-2	NA	NA	NA
#601	27	NA	4'-3	1'-10	5½"	1'-10	4'-3	NA
#701	27	NA	5'-0	2'-0	5¾"	2'-0	5'-0	NA
#702	27	NA	5'-0	2'-2	5¾"	2'-2	5'-0	NA
#703	17	NA	5'-0	2'-2	---	NA	NA	NA

BSF 200/50 REINFORCEMENT EXAMPLE
BSF CONNECTIONS
JVI / PARTEK ØSTSPENN A.S

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JOB NO.
SHEET NO. R200/50

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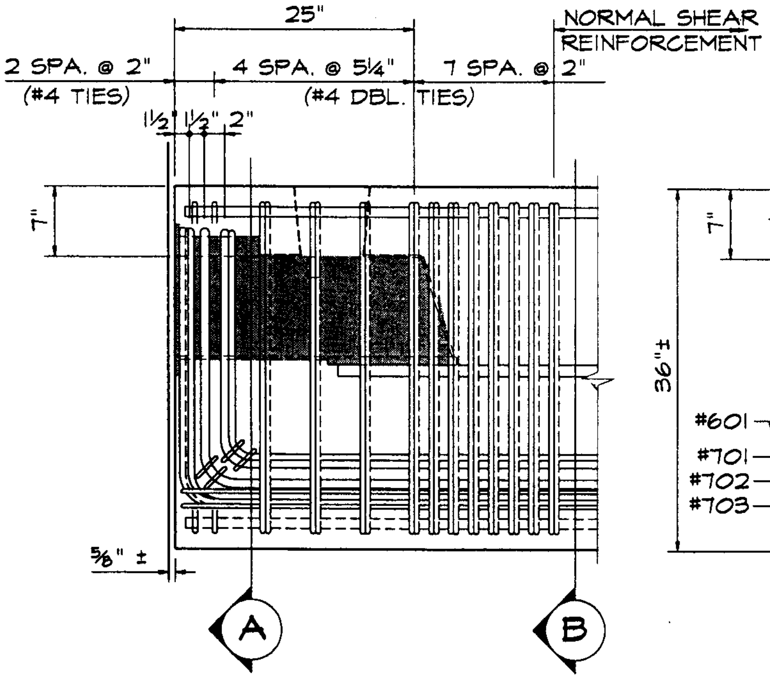
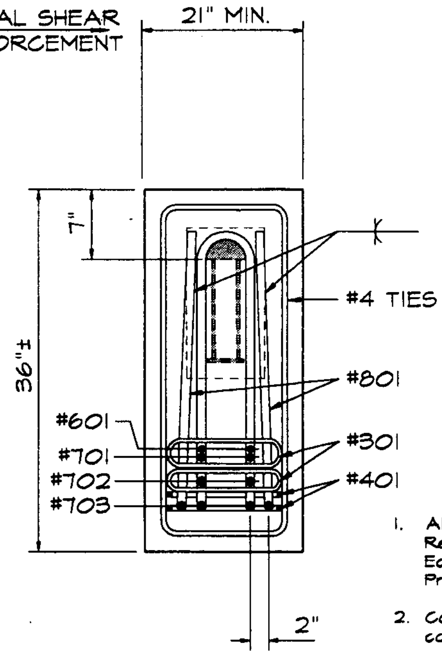
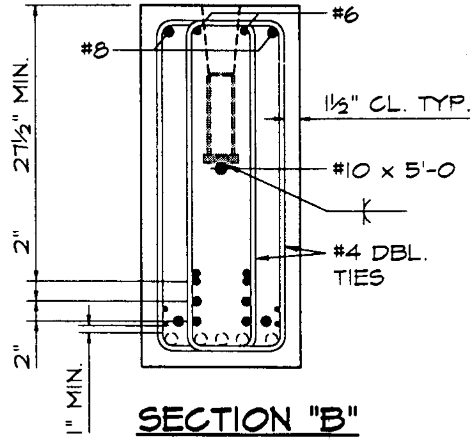


BEAM END ELEVATION

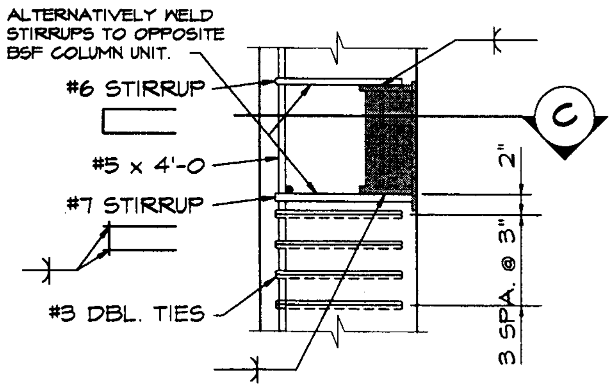


SECTION "A"

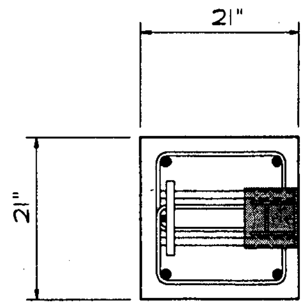


SECTION "B"

DESIGN EXAMPLE - NOT FOR CONSTRUCTION



COLUMN ELEVATION



SECTION "C"

GENERAL NOTES

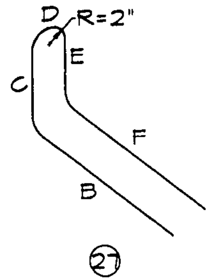
1. All concrete work to be in accordance with ACI 318-89 "Building Code Requirements for Reinforced Concrete", the PCI Design Handbook, 4th Edition, and PCI MNL-116-85 "Manual for Quality Control for Plants and Production of Precast and Prestressed Concrete Products.

2. Concrete to be normal weight (150 pcf) and to have a minimum 28-day compressive strength of f'c = 5000 psi.

3. All welded reinforcing bars and "hanger bars" (#601, #701, #702 & #703) to be ASTM A706 Low-Alloy Steel Deformed Bars (Grade 60). All other reinforcing bars to be ASTM A615-Grade 60.

4. Reinforcing bar welding to be in accordance with AWS D1.4-79 "Structural Welding Code - Reinforcing Steel".

ACI 318 LOAD CAPACITY:
Vertical Design (Ultimate) Strength, ΦRn = 235 Kips (1050 kn)
(1.4D + 1.7L ≤ 235)



BAR BEND SCHEDULE

MARK	TYPE	A	B	C	D	E	F	G
#301	T1	4"	1'-0	3"	1'-0	3"	NA	4"
#401	17	NA	2'-2	1'-0	2'-2	NA	NA	NA
#601	27	NA	4'-10	1'-9¼	5½"	1'-9¼	4'-10	NA
#701	27	NA	5'-10	1'-10	5¾"	1'-10	5'-10	NA
#702	27	NA	5'-10	2'-0	5¾"	2'-0	5'-10	NA
#703	27	NA	5'-10	2'-2	5¾"	2'-2	5'-10	NA
#801	17	NA	6'-6	2'-2	---	NA	NA	NA

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BSF 250/50 REINFORCEMENT EXAMPLE
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JOB NO.
SHEET NO. R250/50

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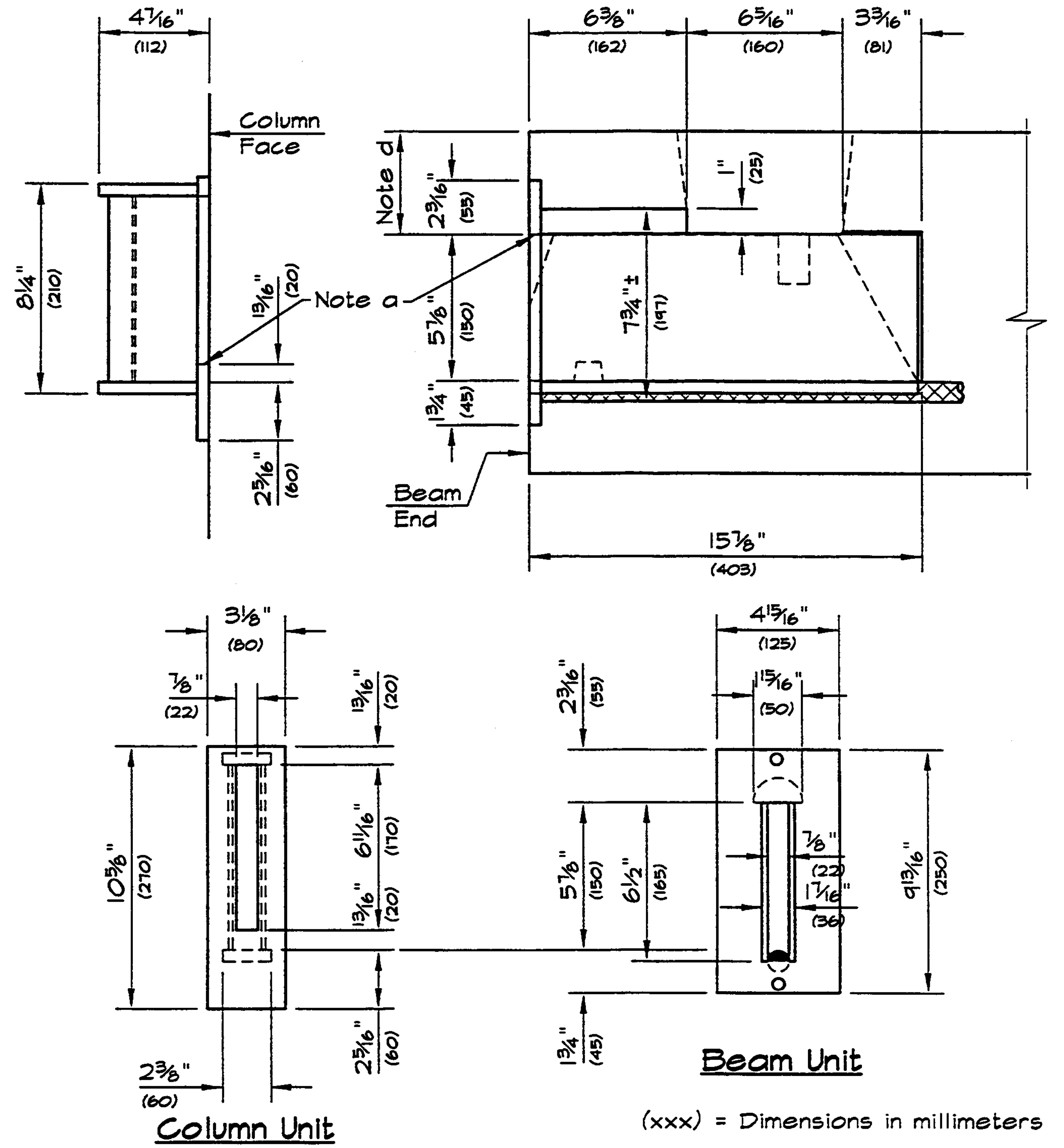
- **BSF UNITS - MAIN DIMENSIONS**



Note a: The arrows indicate the levels that are governing the placing of the units in the forms.

Note b: The dimensions given are for the use of the structural engineer. The production tolerances and provided clearance in the beam unit are not reflected in these figures.

Note c: The beam units are manufactured without the horizontal rebar in the bottom.

Note d: This dimension can be chosen, and is usually standardized by the precaster. The minimum dimension is governed by the size of the stirrups and requirements for concrete cover.

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BSF 150/20 MAIN DIMENSIONS

BSF CONNECTIONS
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2-9-96	SEA	RWK	NTS

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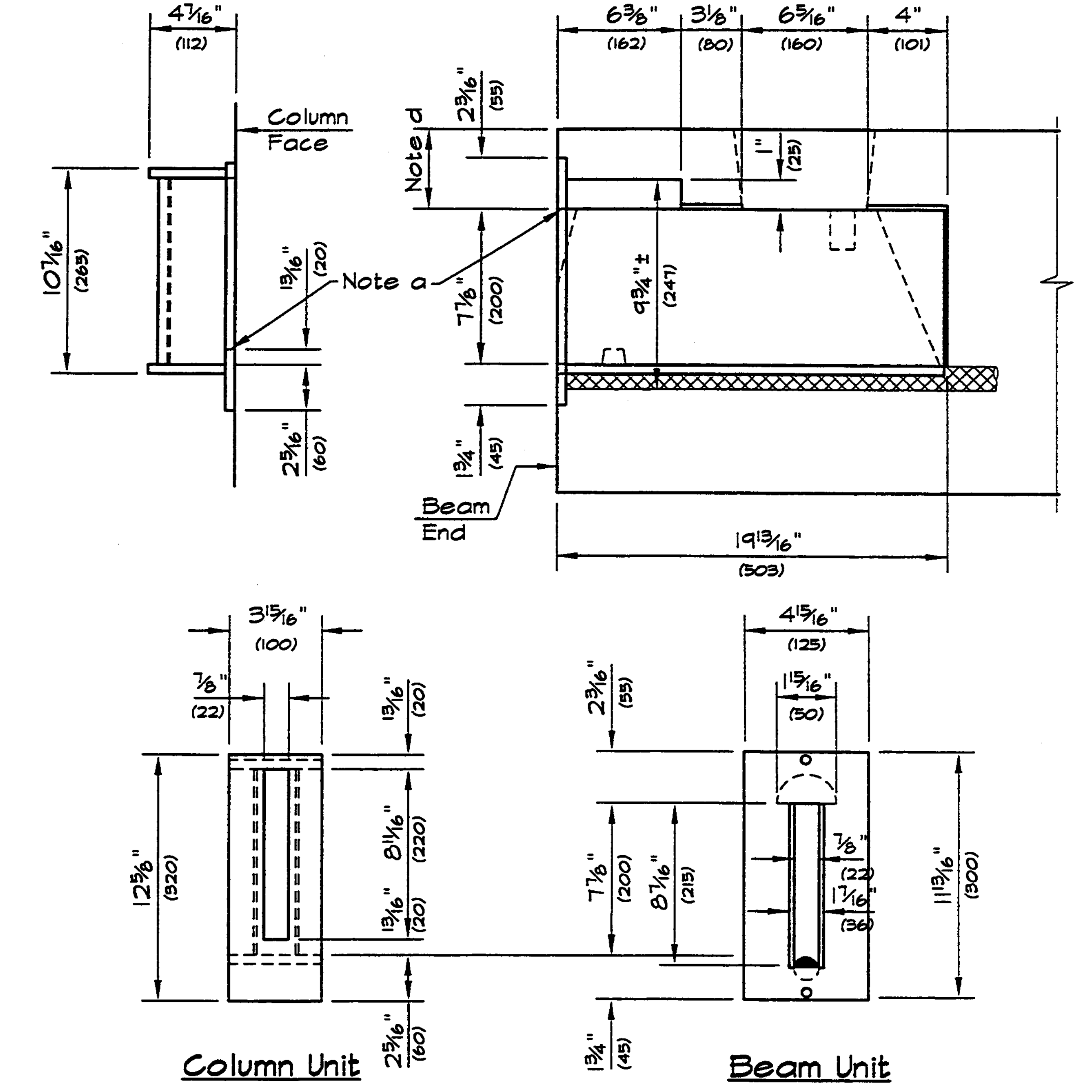

(xxx) = Dimensions in millimeters

Note a: The arrows indicate the levels that are governing the placing of the units in the forms.

Note b: The dimensions given are for the use of the structural engineer. The production tolerances and provided clearance in the beam unit are not reflected in these figures.

Note c: The beam units are manufactured without the horizontal rebar in the bottom.

Note d: This dimension can be chosen, and is usually standardized by the precaster. The minimum dimension is governed by the size of the stirrups and requirements for concrete cover.

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BSF 200/20 MAIN DIMENSIONS

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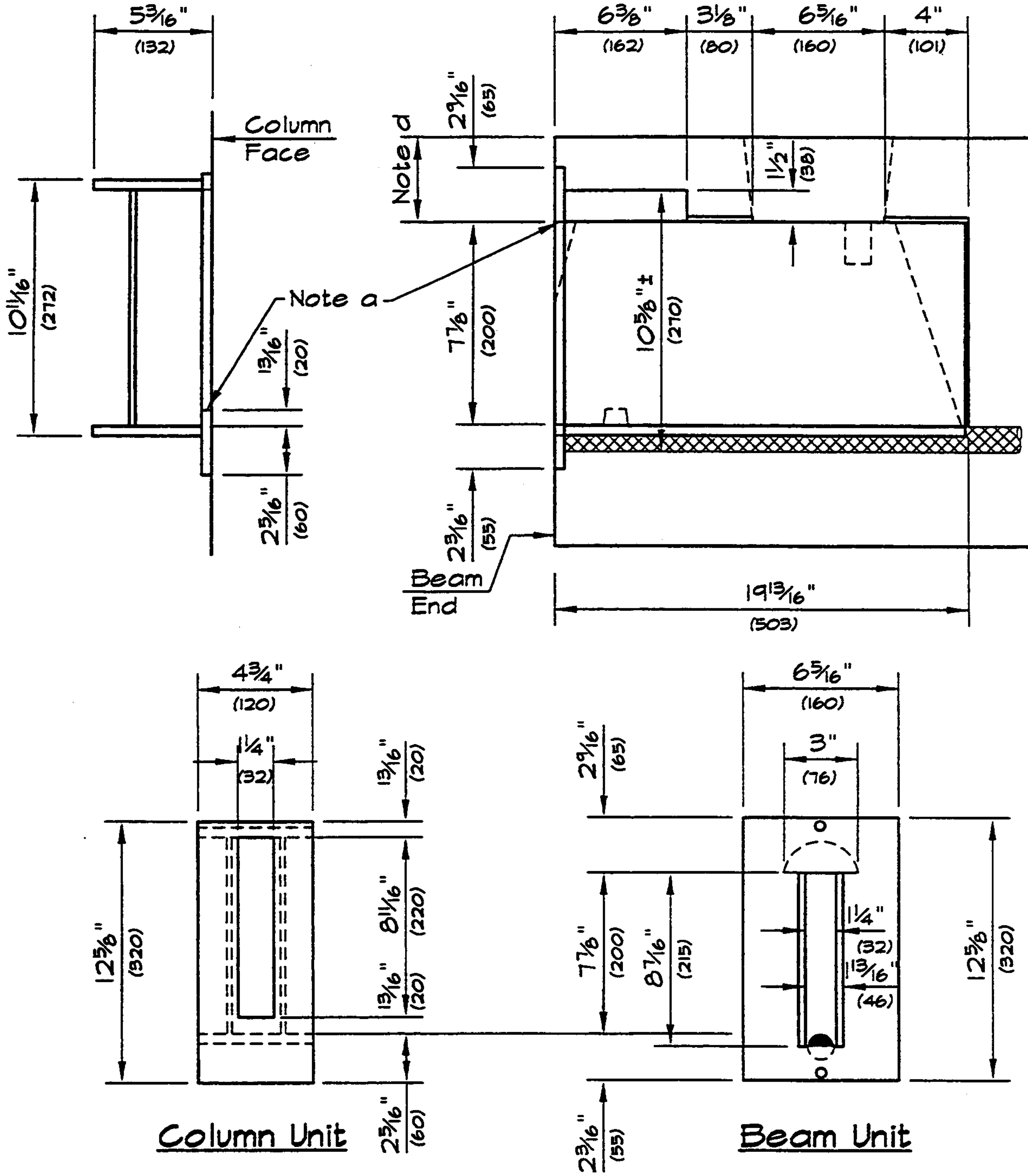

(xxx) = Dimensions in millimeters

Note a: The arrows indicate the levels that are governing the placing of the units in the forms.

Note b: The dimensions given are for the use of the structural engineer. The production tolerances and provided clearance in the beam unit are not reflected in these figures.

Note c: The beam units are manufactured without the horizontal rebar in the bottom.

Note d: This dimension can be chosen, and is usually standardized by the precaster. The minimum dimension is governed by the size of the stirrups and requirements for concrete cover.

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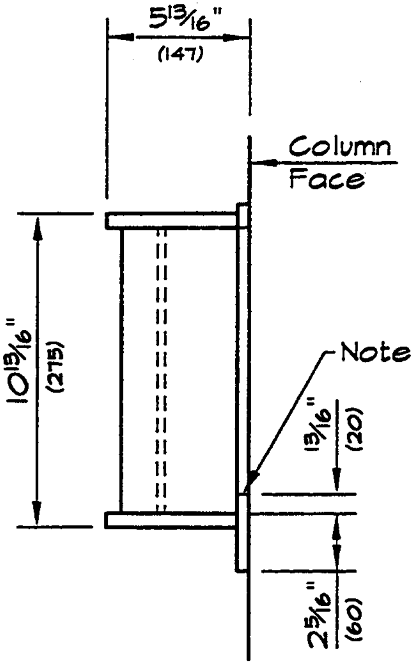
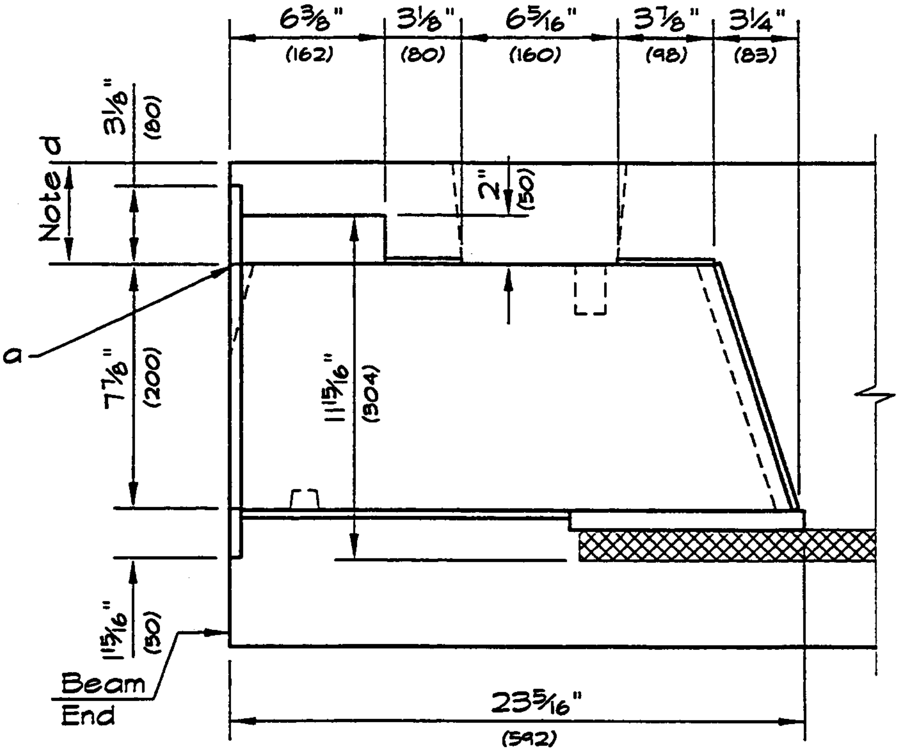
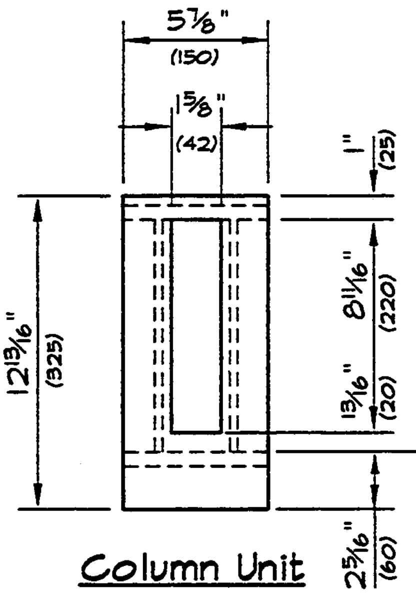
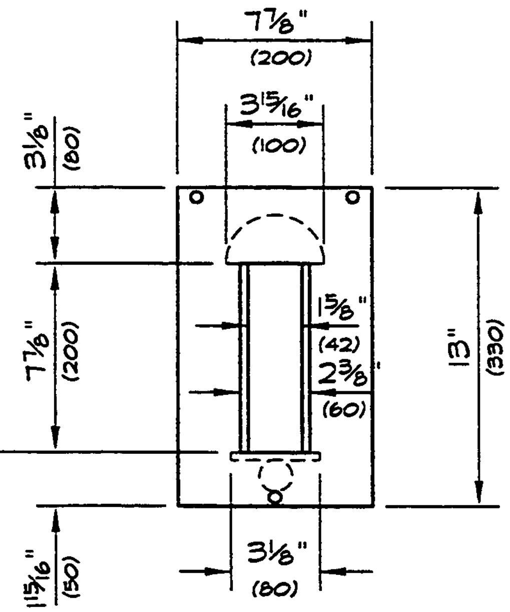






Column Unit



(xxx) = Dimensions in millimeters

Beam Unit

Note a: The arrows indicate the levels that are governing the placing of the units in the forms.

Note b: The dimensions given are for the use of the structural engineer. The production tolerances and provided clearance in the beam unit are not reflected in these figures.

Note c: The beam units are manufactured without the horizontal rebar in the bottom.

Note d: This dimension can be chosen, and is usually standardized by the precaster. The minimum dimension is governed by the size of the stirrups and requirements for concrete cover.

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BSF 200/40 MAIN DIMENSIONS

BSF CONNECTIONS
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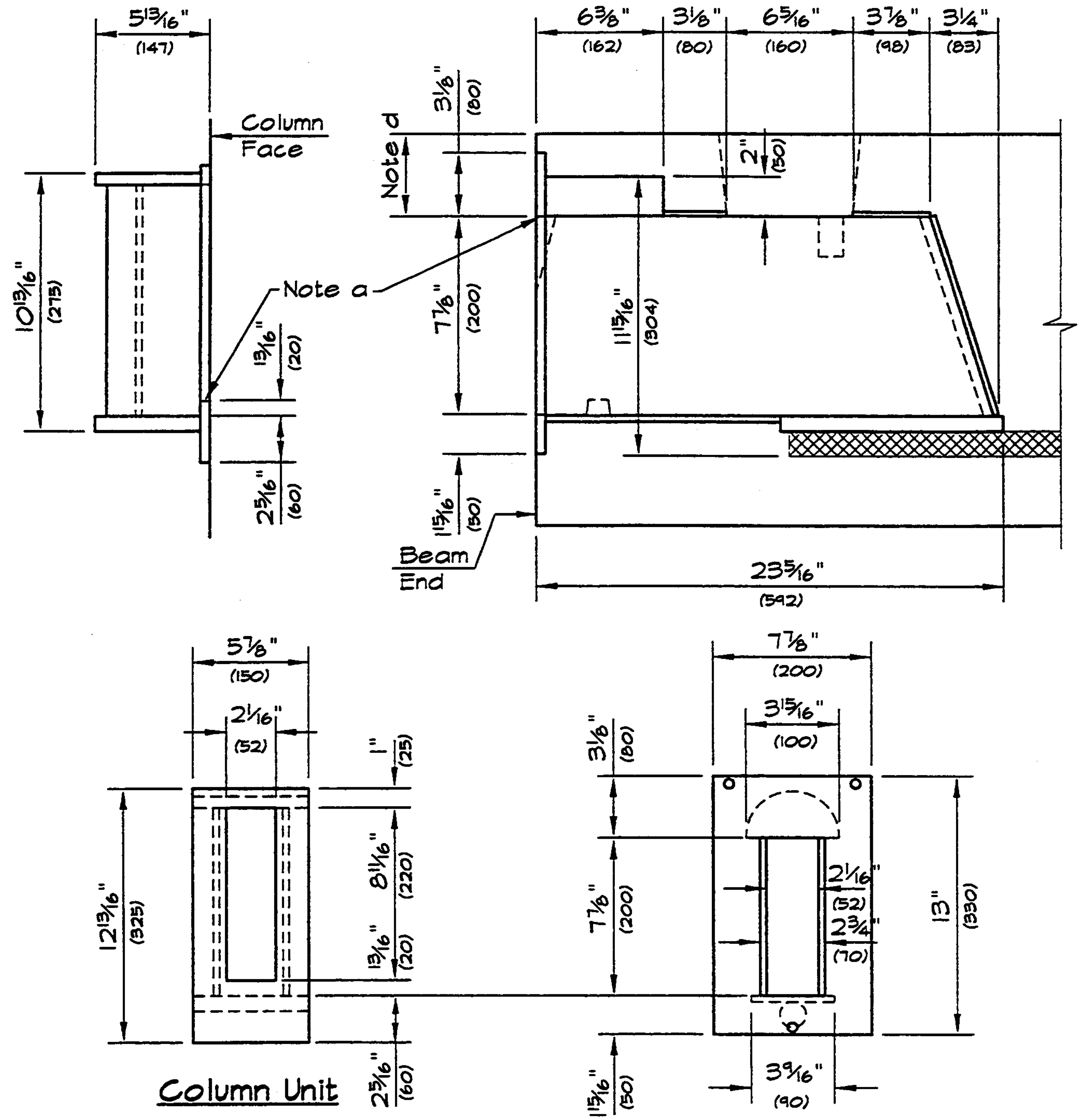

(xxx) = Dimensions in millimeters

Beam Unit

Note a: The arrows indicate the levels that are governing the placing of the units in the forms.

Note b: The dimensions given are for the use of the structural engineer. The production tolerances and provided clearance in the beam unit are not reflected in these figures.

Note c: The beam units are manufactured without the horizontal rebar in the bottom.

Note d: This dimension can be chosen, and is usually standardized by the precaster. The minimum dimension is governed by the size of the stirrups and requirements for concrete cover.

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BSF 200/50 MAIN DIMENSIONS

BSF CONNECTIONS
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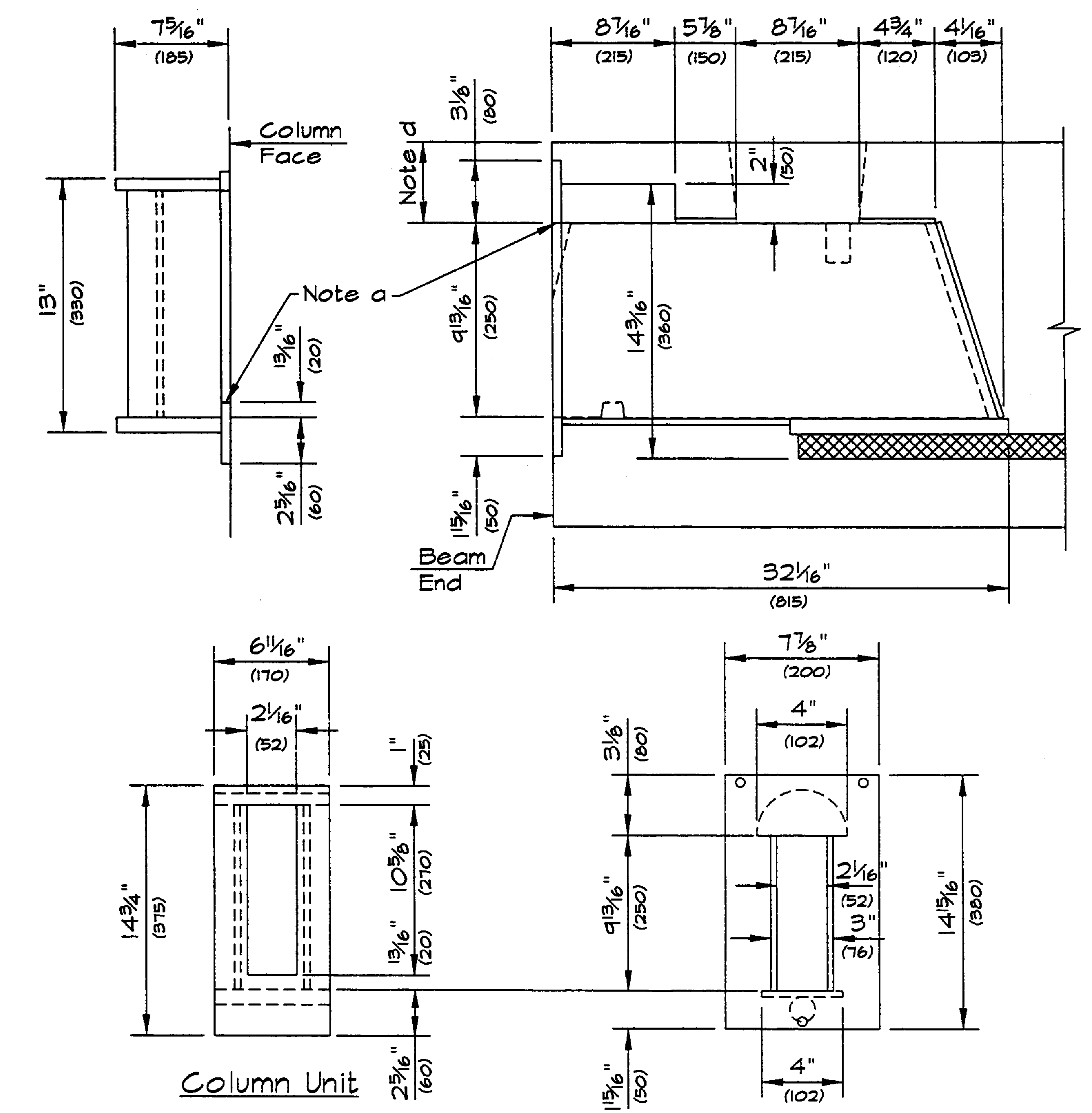

(xxx) = Dimensions in millimeters

Note a: The arrows indicate the levels that are governing the placing of the units in the forms.

Note b: The dimensions given are for the use of the structural engineer. The production tolerances and provided clearance in the beam unit are not reflected in these figures.

Note c: The beam units are manufactured without the horizontal rebar in the bottom.

Note d: This dimension can be chosen, and is usually standardized by the precaster. The minimum dimension is governed by the size of the stirrups and requirements for concrete cover.

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BSF 250/50 MAIN DIMENSIONS

BSF CONNECTIONS
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BSF CONNECTION

CHECKLIST FOR USE OF BSF CONNECTION HARDWARE

1. BSF Connection is a gravity connection only - beam torsion during erection and torsion/axial/lateral forces under permanent conditions must be addressed by the erector and designer (refer to PCI Design Handbook).

2. BSF Connection requires special mild reinforcing in the beam and column elements to transfer forces from steel to concrete - designed by precast concrete manufacturer or designer (refer to example in BSF Technical Data Manual, Part 2 - Engineering).

3. Steel beam and column boxes must be aligned/placed accurately in forms (refer to BSF Technical Data Manual, Part 3 - Fabrication Procedure). Quality Control must monitor placement to accepted industry standards, to reduce/eliminate out-of-tolerance adjustments in field.

4. Erector must be sure that slots in underside of knife engage lower lip of the column box (refer to BSF Technical Data Manual, Part 4 - Erection).

5. Lifting hitches must be located accurately in beam so that beam will be oriented in vertical position when suspended during erection, to ease insertion of knife plate into column box (refer to BSF-Memo 11).

6. Follow BSF-Memo 16 for shop welding of reinforcement (rebar) to bottom of beam box unit.

7. Watch minimum beam sizes for width and depth as shown in BSF Technical Data Manual, Part 1, for rectangular beams. Increased/decreased beam widths are possible with shallower/deeper beam depths, and/or higher concrete strength (refer to design example in BSF Technical Data Manual, Part 2 - Engineering).

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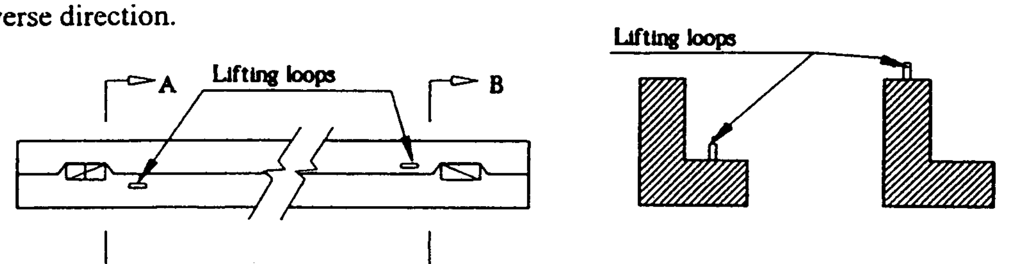
2/16/96	RRO	RWK		BSF CONNECTIONS JVI / PARTEK ØSTSPENN A.S

PLUMBING OF BEAMS

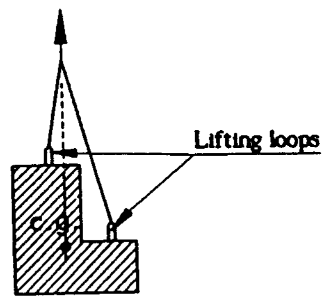
It is important that the beams are vertical when suspended from the lifting loops or any other lifting devices, in order to ease the erection process. For beams with unsymmetrical cross-sections - as ledge beams - it may not be obvious how this best can be achieved.

One method is shown in the figure below. In this case the lifting loops must be placed symmetrically with respect to the centre of gravity of the beam - both in the longitudinal and transverse direction.

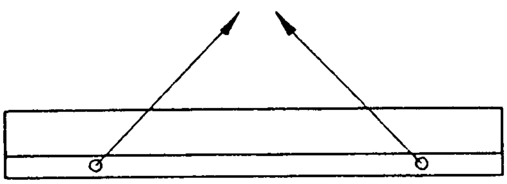


Plan view Section A-A Section B-B

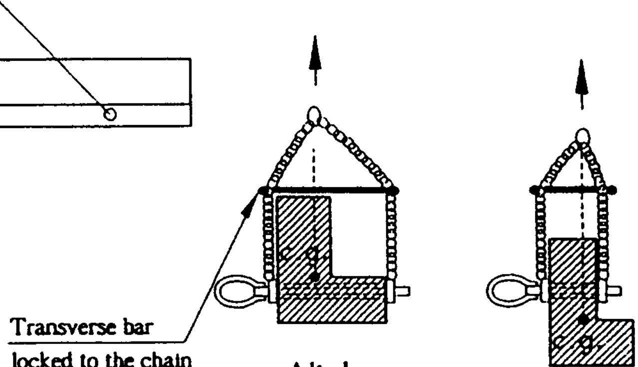
It is also possible to place two lifting loops in each end of the beam, as shown in the figure to the right. Lifting directly above the centre of gravity is then achieved by adjusting the lengths of the straps.



A third possibility is to make two horizontal holes through the beam. Steel rods can be inserted in these holes and used to hook on to.
Plumbness of the beam can then be achieved by adjusting the lengths of the chains above the transverse bars over the beams. The transverse bars are there to prevent the chains from damaging the edges of the beam. The chains must be of the same length from these distance holders down to where it is hooked on to the steel rod, so the distance holders should lock to the links in the chain.



Side view



Alt. 1 Alt. 2

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PLUMBING OF BEAMS

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BSF-MEMO 11

WELDING PROCEDURE FOR BEAM UNIT REINFORCEMENT

This memo describes the welding procedure for the reinforcing bar T4 (see MEMO 1) for beam units without bottom plate.

Some beam units do not have a bottom plate in order to reduce the cost of the units, and to make them less spacious. In these cases the tensile bar T4 must be welded directly to the side plates.

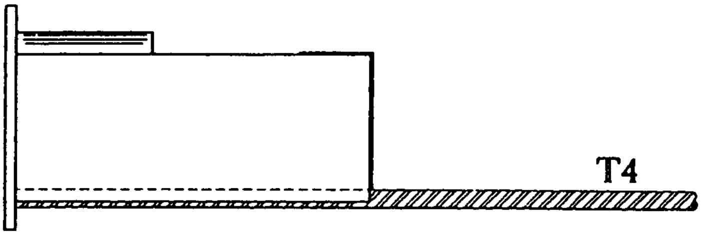

Figure 1

Method:
A suitable guiding plate (for example of plywood) is prepared (see figure 2). The plate must be approximately 100 mm longer than the space in the beam unit, with a cut out for easy gripping. The height must be 2 mm more than the height of the knife, and the thickness equal to the thickness of the knife - or a little less.

Figure 2

The beam unit is placed upside down, the guiding plate is placed inside the beam unit, and the reinforcing bar T4 is positioned at the top of the guiding plate (see figure 3).

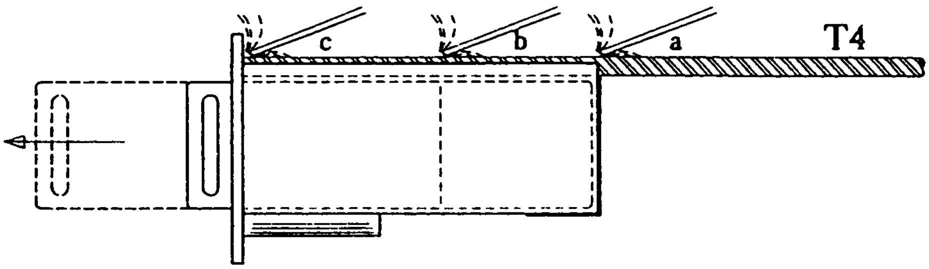

Figure 3

The reinforcing bar T4 shall then be spot welded in three locations (a, b and c, see figure 3) to the side plates, on both sides, starting at location a. The guiding plate must during this process be gradually withdrawn from the beam unit in order not to get stuck due to shrinkage of the welds. The welder must take the necessary precautions to ensure that there will be sufficient space between the side plates so that the knife later can be inserted without any problems. This is especially important when welding at point b, if necessary a tool must be used to bend the plates apart while welding at this point.

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WELDING PROCEDURE FOR BEAM UNIT REINFORCEMENT

BSF-MEMO 16

WELDING PROCEDURE FOR BEAM UNIT REINFORCEMENT

After spot welding the guiding plate is fully removed. The full length of the welds on both sides can now be completed. Alternate welding on both sides is necessary to prevent longitudinal bending of the beam unit during this operation.

To prevent welding splutter from getting into the beam unit - which will make the insertion of the knife difficult or impossible - it is advisable to keep the welding rod at approximately 20°, as shown in figure 4.

Extra welding or silicone should be used at locations a and c to make sure the beam unit is water tight.

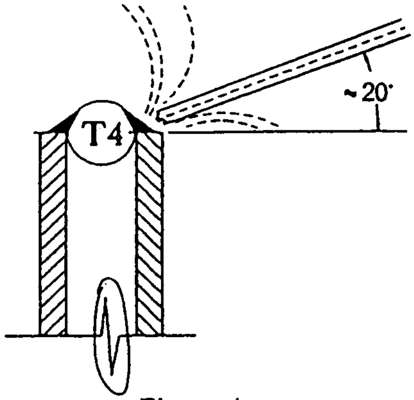

Figure 4

Always make trial insertion of the steel knife into the beam unit before it leaves the welding shop.

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WELDING PROCEDURE FOR BEAM UNIT REINFORCEMENT

BSF-MEMO 16

PART 3 - PRODUCTION/FABRICATION

GENERAL

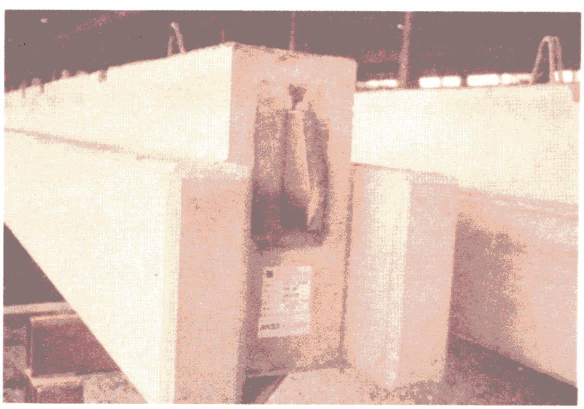
The BSF connection system eliminates the need for large projecting concrete corbels which disrupt column production forming and complicated reinforcing shapes. Eliminating alterations of the forms saves labor and speeds production.

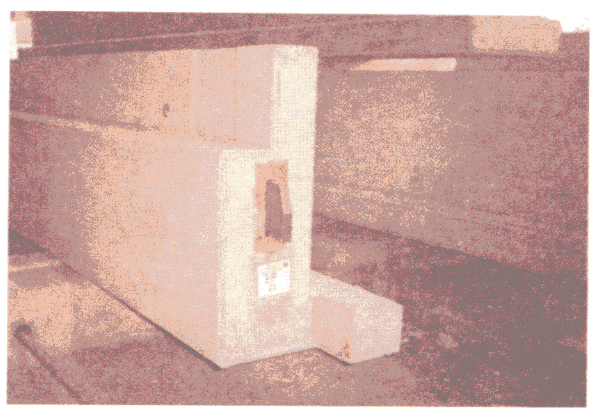
Flexibility in column production is increased with the BSF connection system as the columns can be cast in the erection sequence rather than being dictated by locations of projecting corbels.

As the various steel components of the system have a narrow fit, attention has to be paid to tolerances during fabrication. In order to facilitate this, a few simple devices and methods have been developed that ensure correct and secure placement of the BSF box units during casting. It is also of importance that the forms are sufficiently stiff to prevent deflections during casting that may alter the position of the units beyond acceptable limits.

Attention also has to be focused on the placement of lifting inserts in the beams, as it is important that the beams hang vertically during handling.

The reinforcement that is cast in the concrete can be standardized, and as a result there are less chances of errors during design, drawings and production. This also provides great advantages in any CAD-system.





FABRICATION PROCEDURE

The BSF connection system, consisting of a column unit, a beam unit, and a knife (that fits into the beam unit) are delivered separately from the manufacturer, normally without any reinforcement welded on, for convenience and economy in packing and shipping (Photograph 1). The reinforcement according to the production drawings is then welded on to the column and beam units, after which they are brought to the plant production area (Photographs 2 and 3).

The casting procedure for the BSF system requires placing of the column and beam units ***accurately*** in the forms. The knife is not inserted into the beam unit until after the beam is cured and stripped from the form (Photograph 4).

The method chosen for placing the beam and column units consists of guide pieces positioned in the forms. They can be made of any suitable material, for example steel, plastic or plywood. However, steel is recommended, when casting large series (Photograph 5).

It is very important always to use guiding-pieces. They fix the position of the openings in the column and beam units. This reduces deviations and ensures available tolerances for the location of the knife in the completed connection.

BEAM UNIT

The guiding-pieces are fastened ***securely*** to the end plates of the beam form (use screws). Make sure that the distance from top of the beam to the top of the guide piece (**a** on Fig. 1) is accurate. This is where the beam rests on the knife after erection, and a correct position of this point reduces the vertical deviation of the beam unit. Oversize holes are drilled in the form end plate to match the threaded holes in the front plate of the beam unit. ***When fastening the beam unit to the form end plate, make sure that the top of the opening in the beam unit is in contact with the top of the guiding-piece (Photographs 6 and 7)***.

The reinforcement-cage containing the beam units with end plates is lifted into the form and adjusted into correct position. Before casting, a specially made tapered plug is placed into position in the top opening of the beam unit (**b** on Fig. 2). This plug is removed from the beam after curing of the concrete to form the BSF access pocket. It can be made of any suitable material, but steel is recommended (Fig. 3 shows a cross-section of a removable plug made of steel). After curing of the concrete and stripping, the beam is transported to the storage yard. Here the knife with the lockpin is placed into position (Photograph 4).

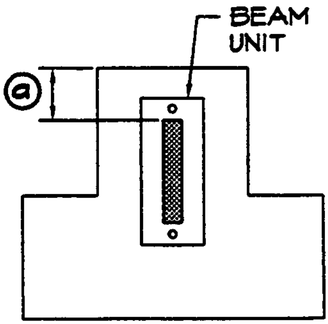


Figure 1
Beam End View

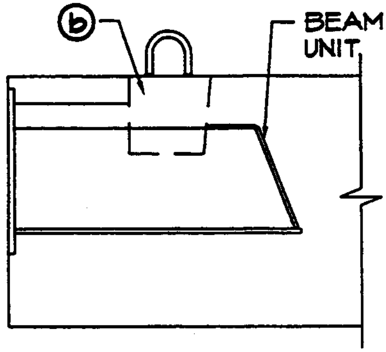


Figure 2
Beam Side View

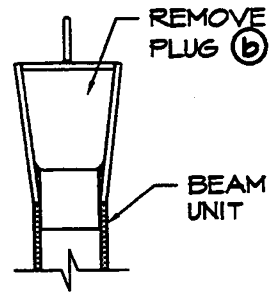


Figure 3
Tapered Plug

COLUMN UNIT

Columns are normally cast horizontally in steel or wood forms with the guide pieces fastened to the form according to the production drawing. Measurements are given from bottom of column to the lower end of guide pieces. In practice it is difficult to measure to the support plate for the knife inside the column unit, so the latching edge 20 mm above is chosen (**c** on Fig. 4). For correct placing of the guide piece, it is recommended to use a plywood guiding plate (Photograph 8). Note the line on the plate corresponding with the lower edge of the guide piece. This line is used when measuring from bottom of column to the lower end of the guide piece. The picture shows a steel form, and here it is convenient using spot welding to fix the guide piece. With plywood molds, use screws. When placing the column unit in the form, make sure that there is contact between edge (**c** on Fig. 4) and the guide piece.

The BSF-system allows column units to be located on all four sides of the column at the same level, but in practice there are very seldom more than three units (Photographs 9, 10, and 11). It is best to use bottom and side walls of the form for placing the column units, top face should only be used when there are four at same level. A special jig must then be made on top of the form for fixing the fourth column unit.

Fixing a single column unit to the bottom of the form is easily done using silicone around the rim of the unit and pressing it onto the form. Fixing on one side can be done using an adjustable bolt (Photograph 12). When double-sided, the units are welded together (with standard reinforcement) in suitable jigs. In both cases the column units are held tightly in position with help of the pressure from the sides of the form.

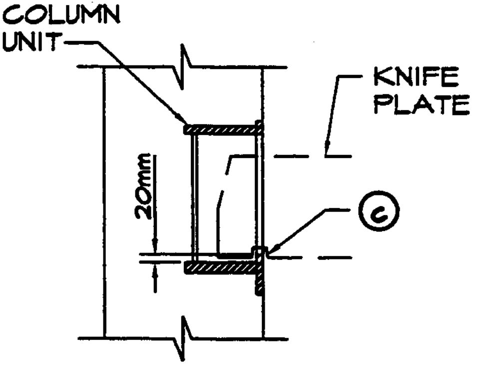


Figure 4
Column Side View

The procedures described above for the beam and column units are very simple, but very important. Practice has shown that they ensure a good result, and the number of errors are minimized. Correct positioning of the guide pieces assures the correct placement of the steel knife in the completed connection.

ADVANTAGES

The advantages of the BSF connection system over conventional column corbels and dapped beam ends include the following:

- Time saving in design and drafting
- Increased quality assurance in design, drafting, and production
- Reduces time for altering column forms
- Continuous steel forms for production efficiency
- Flexibility in column production schedules
- Cost savings

TOLERANCES

The BSF connection units are subject to accepted Industry Standards for fabrication and erection as defined in Part 4 - Erection.

The tolerances for placement during fabrication must be closely monitored to ensure that additional field adjustments are allowed during erection. The nature of the steel beam and column boxes lend themselves to accurate placement in the forms during fabrication and are easily checked by quality control inspection procedures prior to casting.

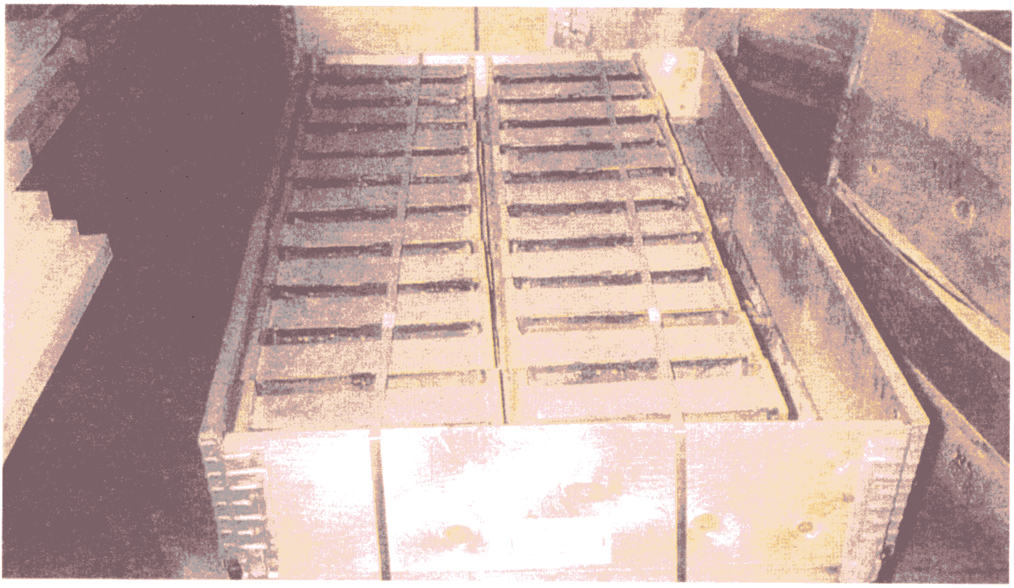
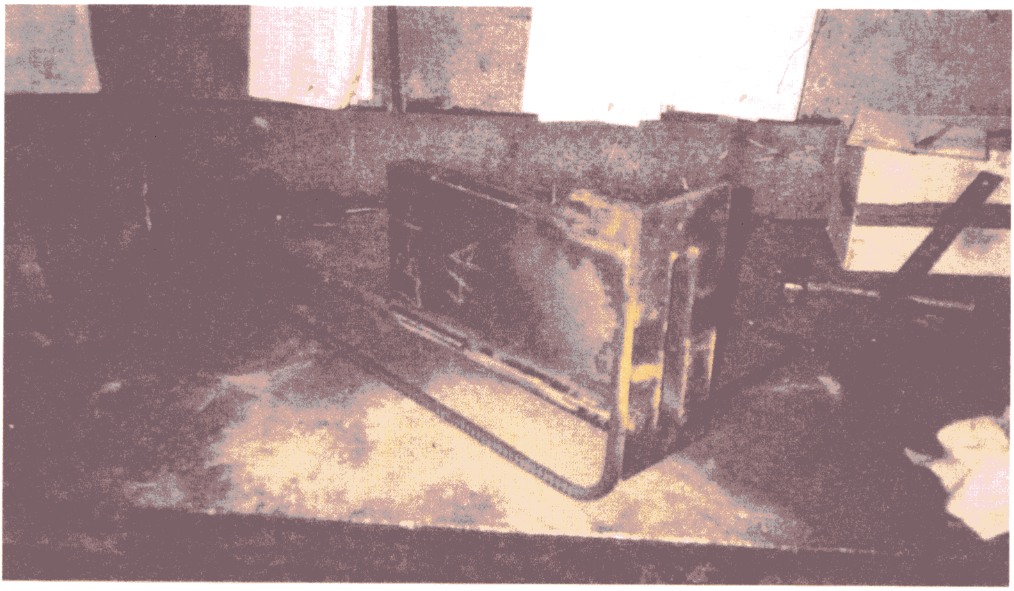


Photo 1



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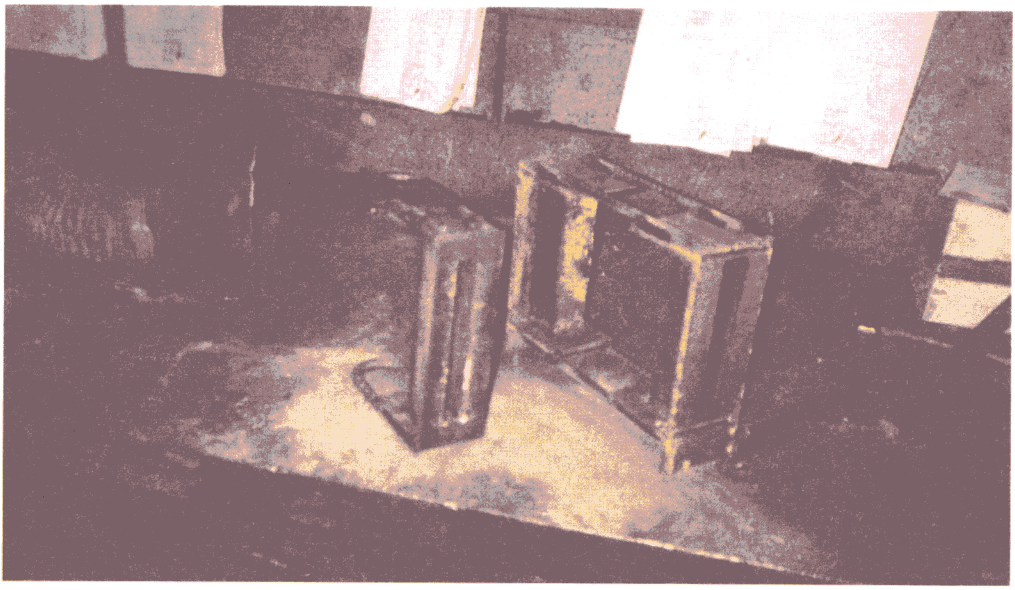

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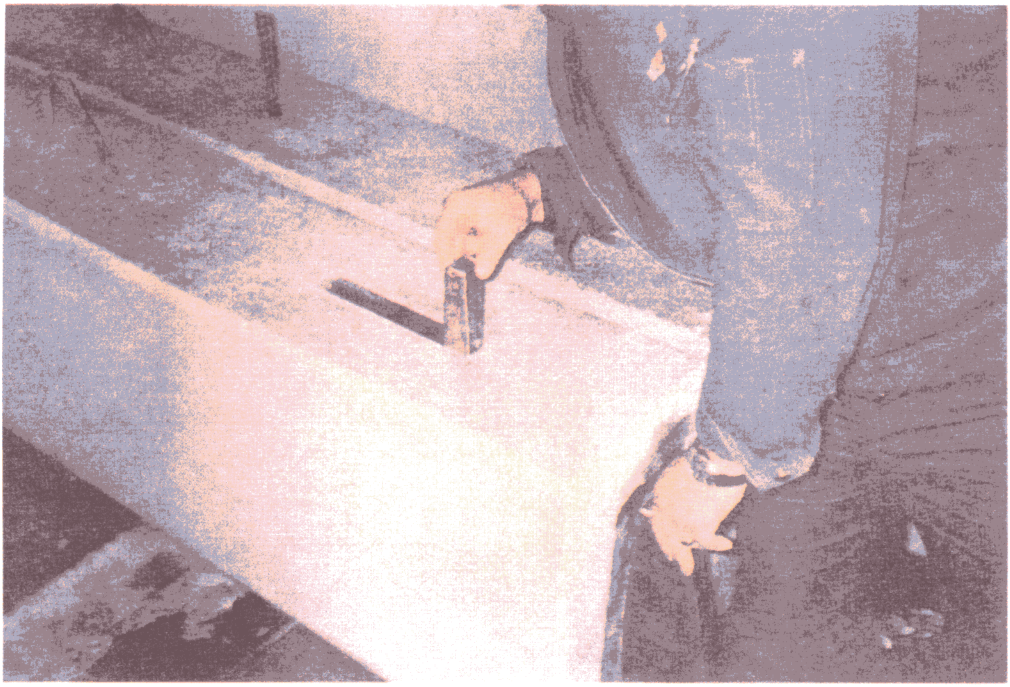

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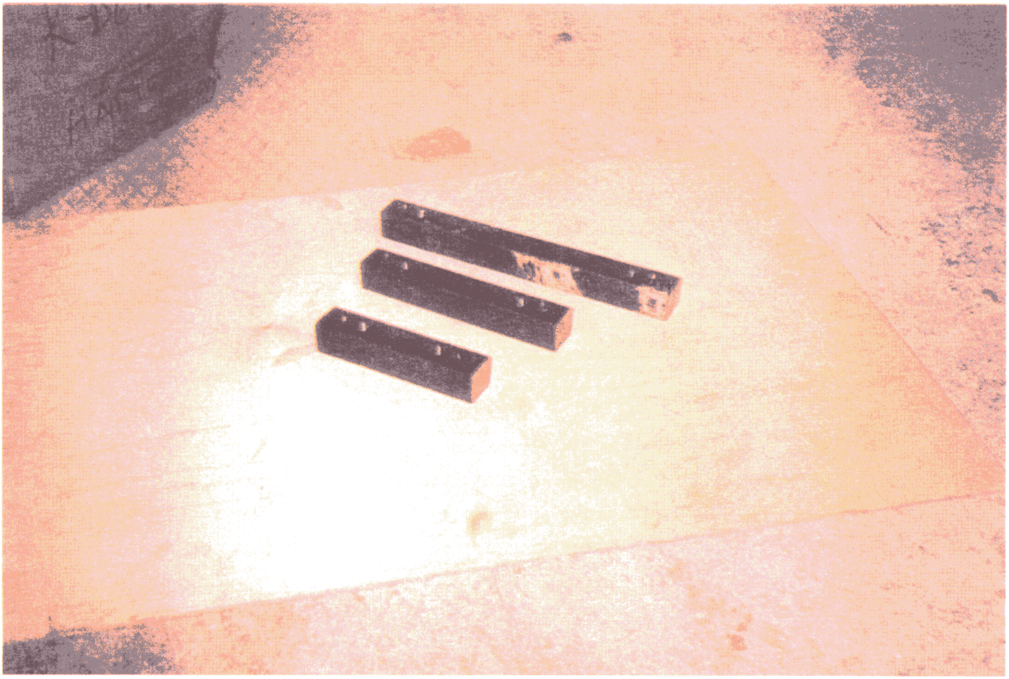

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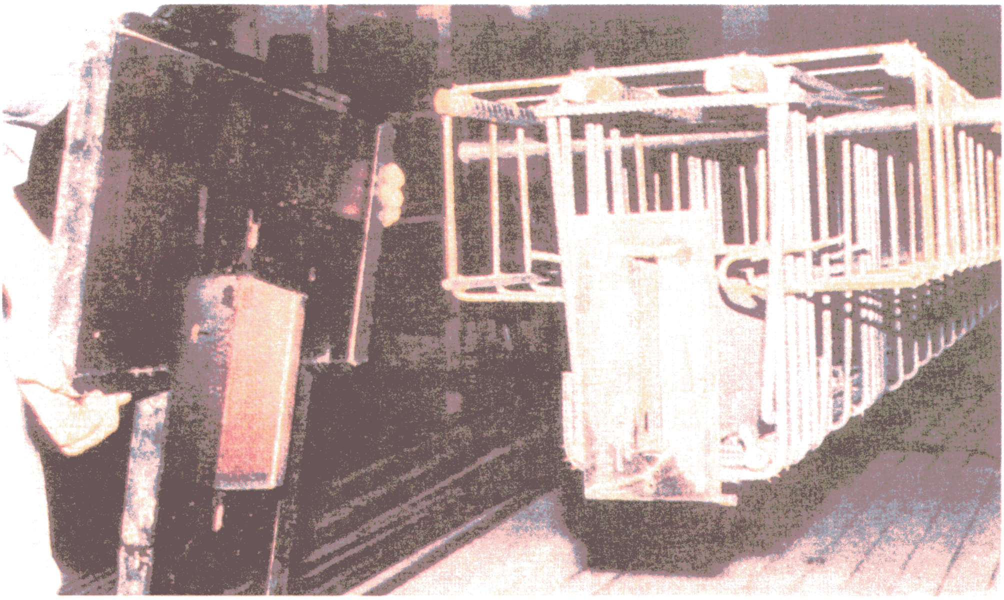

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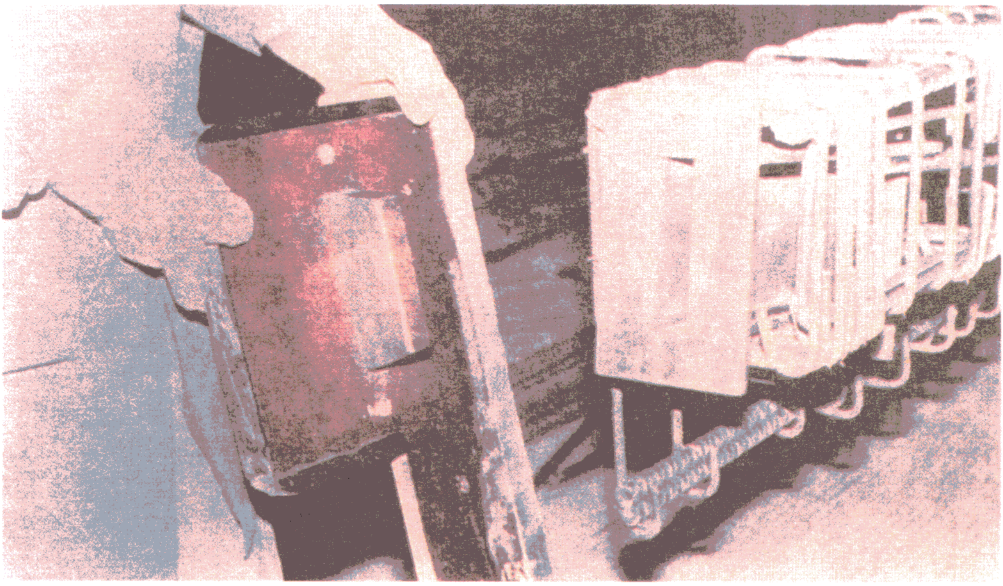

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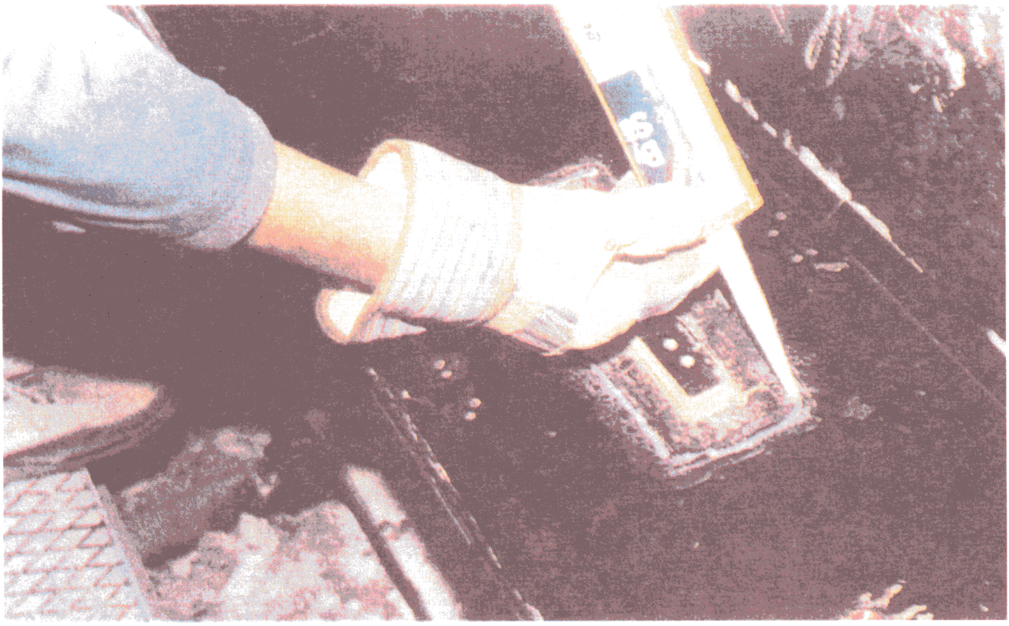
Photo 8



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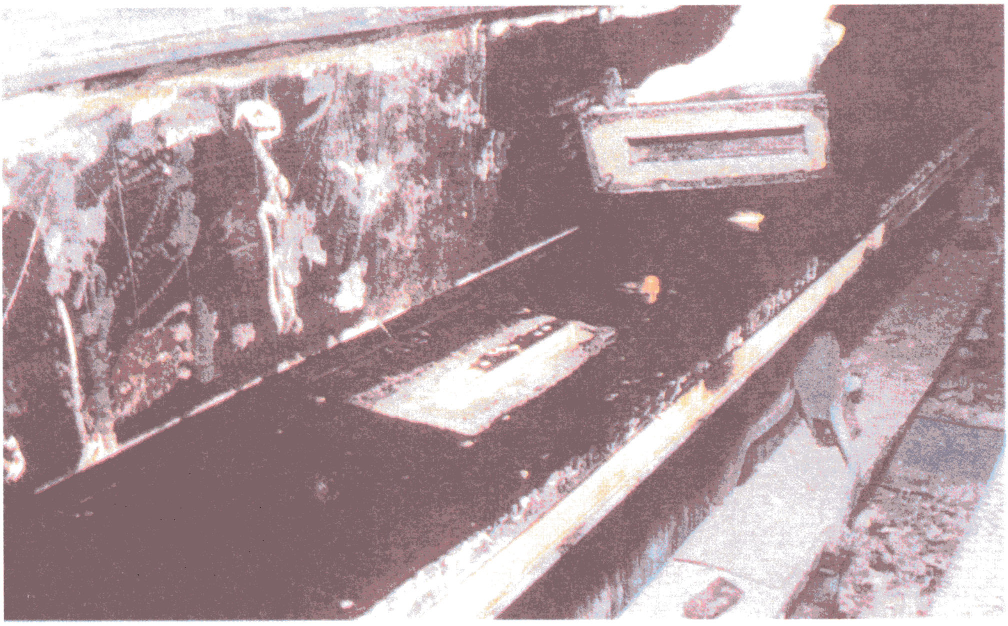


Photo 10

Photo 11

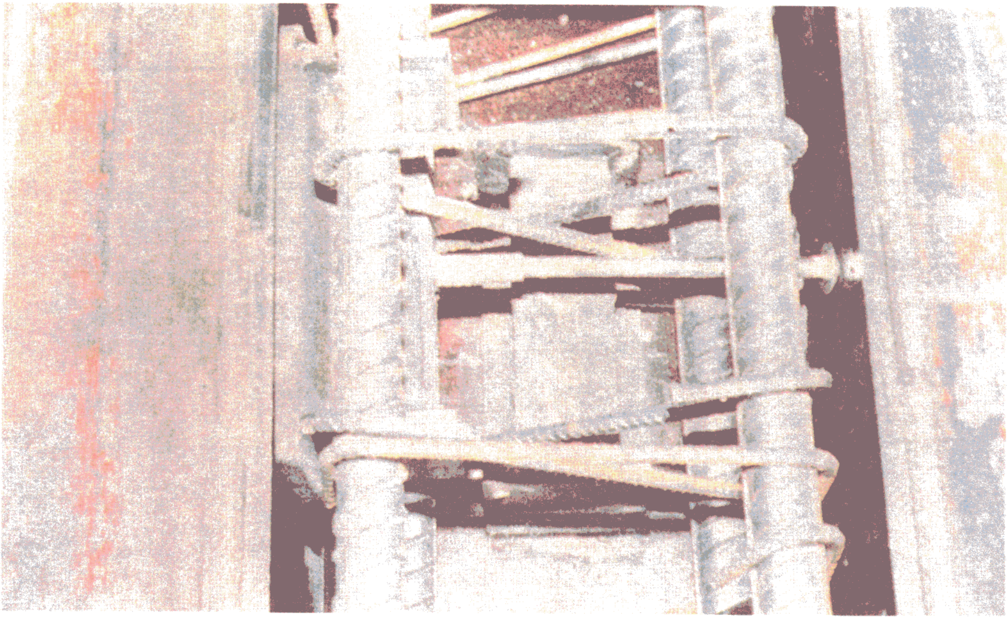


Photo 12

Photo 13

ERECTION

After a beam is hoisted into position (Figure 1), with the BSF knife plate fully retracted in the beam box, the BSF knife plates are then extended with a steel crowbar into the column-box until they hit the back wall. The beam is then lowered slowly to its final resting position, making sure that the slots on the underside of each knife plate engaged with the lip of the column box. While this operation can be done by one person, it is more convenient and quicker with two.

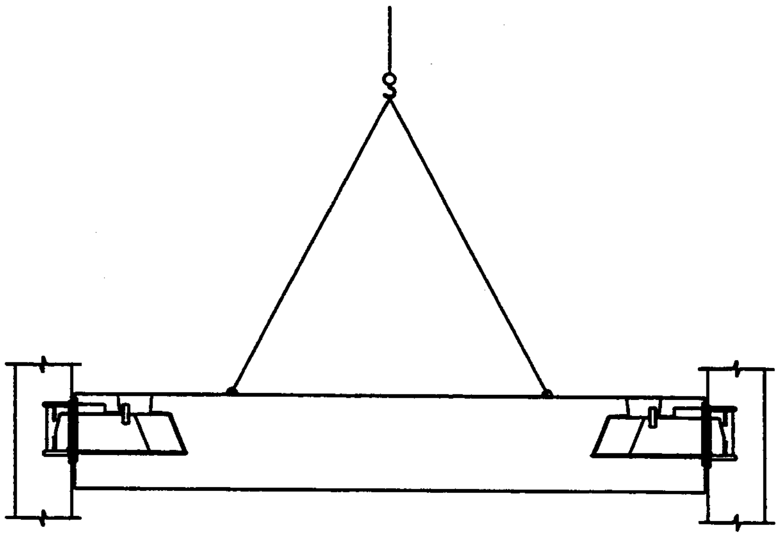


Fig. 1

Before removing lifting-cables from the beam, the joint widths are evaluated (Figure 2). Normally this joint should be 5/8 inch, but production and erection tolerances can cause variations between 0 and 1-1/8 inch. Joint-widths adjustments are made the following way, again using the crowbar:

Position A: Increase Joint Width

Position B: Decrease Joint Width

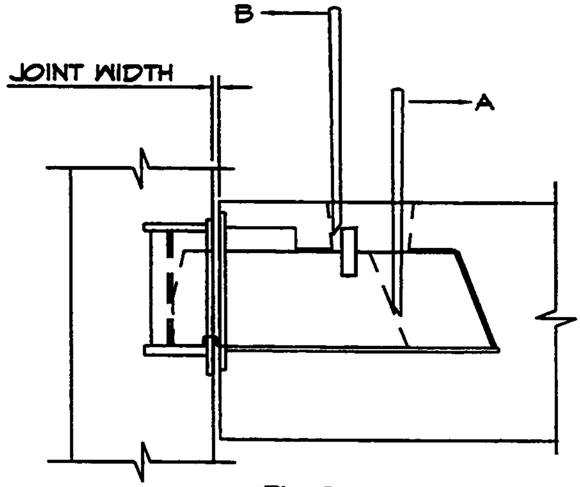


Fig. 2

TORSIONAL CONSIDERATIONS

During erection, torsional forces are created in the beam as deck members are erected on one side of the beam or if deck members on either side of the beam vary in length. These torsional forces must be considered in the planning of the erection process.

Normally, the beam requires temporary torsion support until the permanent and monolithic connections of the deck (hollow core slab or double tee) with the beam are completed. If needed, the torsional movement at the joint can in most cases be locked by installing temporary support posts or column brackets, or by using quick setting grout in the joint between the beam and the column before the deck members are erected.

BEAM END JOINTS

This joint can be sealed with concrete grout or ordinary joint sealant. If fire protection is required, there are two ways to cover and protect the part of the knife plate that is exposed in the joint:

- Fill the joint with mineral wool before applying a fire-resisting joint sealant.
- Grout the entire joint, and use a polyethylene strip pressed into the joint at the sides and underneath to prevent leakage during grouting.

TOLERANCES

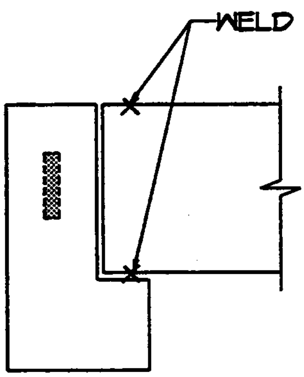
The BSF connection units are subject to accepted industry standards for fabrication and erection. The BSF connection units allow for the following out-of-tolerance adjustments in the field:

Horizontal Length (Axial):	Joint Opening 0 - 1-1/8 inch (30 mm)
Vertical:	Up or Down 3/4 inch (20 mm) - adjustment up requires adding shims; adjustment down requires cutting (milling) bearing nose of knife plate
Horizontally:	BSF 150/20 Plus/Minus 3/16 inch (5 mm) BSF 200/20-50 Plus/Minus 3/8 inch (10 mm) BSF 250/50 Plus/Minus 3/8 inch (10 mm)
Angular Tolerance:	BSF 150/20 Plus/Minus 2 Degrees BSF 200/20-50 Plus/Minus 3 Degrees BSF 250/50 Plus/Minus 3 Degrees

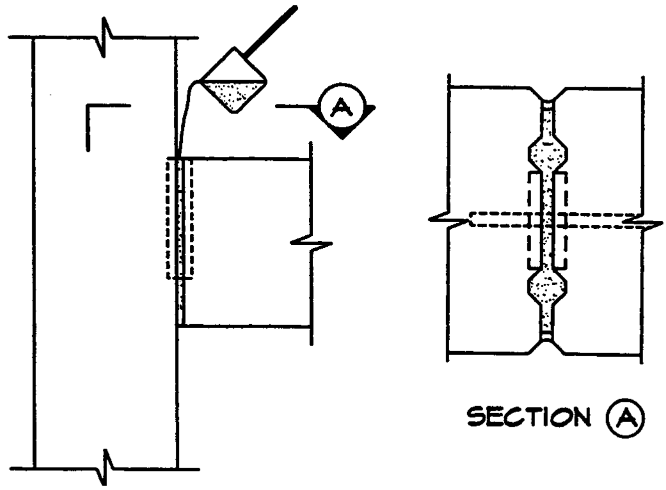
NOTE: Field modifications to the column box (cutting face plate) or knife plate (milling of bearing nose), or adding shims allow for out-of-tolerance adjustments in the field.

BEAM TORSIONAL STABILITY DURING ERECTION CAN BE ADDRESSED IN SEVERAL WAYS

Beam/Deck Permanent Connections
Grouted Beam/Column Joints (Temporary Torsion Support)
Temporary Support Posts
Temporary Steel Brackets Mounted on Columns
Temporary Beam/Column Torsion Connections

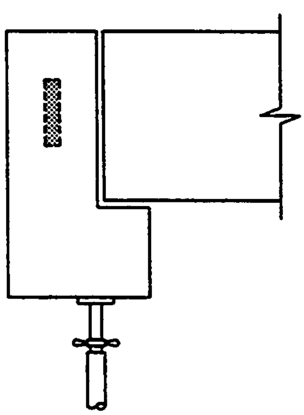


Beam/Deck Permanent Moment Connections

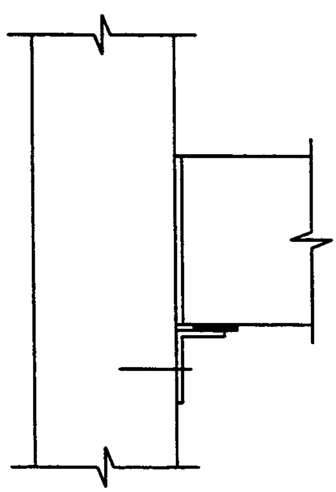


Grouted Ends
(For Temporary Torsion Support - Shear Keyways ("ducts") in the Beam End Face and Column Face Required)

NOTE: Permanent beam/deck connections required



Temporary Support Post



Temporary Steel Bracket

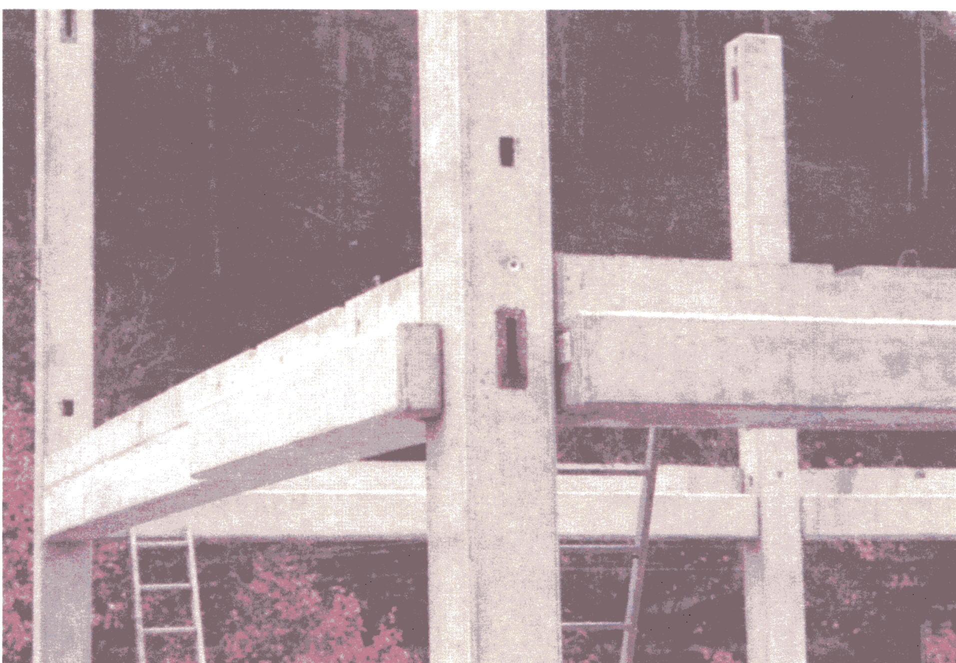
PERMANENT CONNECTIONS

The BSF connections are intended to be a vertical gravity connection and torsional forces imposed on the beam must be resisted by the permanent and monolithic connection of the slabs (hollow core slab or double tee section) with the beam or by beam to column connections.

Although the steel support knife plates can resist some torsional forces, the BSF connections have not been designed or tested for torsion. Also, due to the allowable tolerances in the beams and column box units, any allowed torsion movement will cause a visible inclination of the beam. The beams must be supported against torsion by temporary posts (struts), temporary column brackets or quick setting grout in the beam/column joint during erection. After field welding of permanent connections and/or casting of monolithic concrete between the beam and deck, torsion to the BSF connection will be eliminated.

For further information on the analysis and design of precast , prestressed concrete structures and components including erection bracing and torsion, refer to the PCI Design Handbook, Fourth Edition, 1992. Specifically, Chapters 3, 4, and 5 provide a thorough and detailed explanation on the concepts and design requirements of precast, prestressed concrete components and their connections.

The BSF connection system is a standardized beam end gravity connection that facilitates the production and erection process for precast concrete structures, and can be easily incorporated into the design. The Engineer-of-Record's role regarding design of permanent connections for lateral forces and structural stability of the building frame remain unchanged. Refer to Part 2 - Engineering for further discussion of the Designer's Role and Responsibility.



PART 5 -SPECIAL REQUIREMENTS

The BSF connection is also capable of accommodating various special project requirements. The following is a partial list of solutions that can be utilized.

FILLING JOINTS FOR FIRE PROTECTION

The joints between the columns and beams are normally filled so that the BSF connections have protection and will not be in the critical component when considering fire protection ratings. Various materials may be used to fill the joints between the beams and columns:

When allowance is made for movement in joints, use self-expanding fire-resistant foam or mineral wool.

Most commonly used is a quick setting grout. For damming of the mortar, round rubber/ plastic backing is needed, such as TREMCO joint backing, obtainable in diameters from 3/8 to 2 inches (10 to 50 mm).

A joint-backing of suitable diameter is placed along the outer rim of the concrete-joint between the beam-end and the column (Example: with 5/8-inch (15 mm) opening, a 13/16-inch (20 mm) joint-backing is suitable.) A suitable pre-mixed dry mortar is mixed and poured into the joint (Fig. 1). After curing of the mortar, the joint-backing is pulled out leaving a 5/8-inch to 3/4-inch (15-20 mm) deep recess.

To improve the joint appearance, elastomeric joint-sealant may be used as a finish.

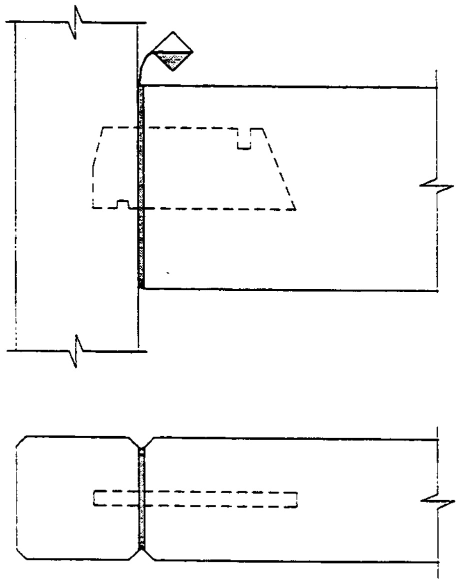


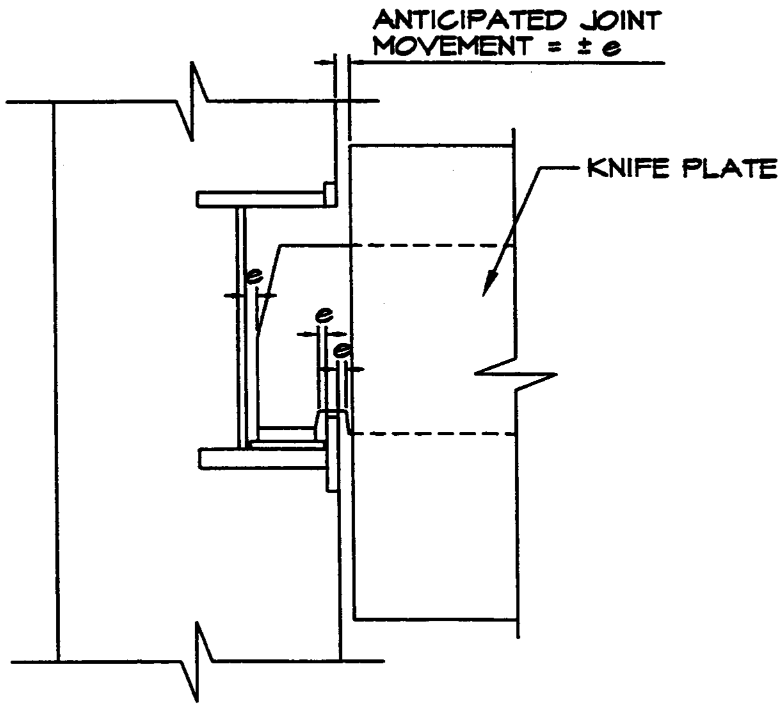
Fig. 1

NOTE: Knife blade can be zinc coated for corrosion protection and elastomeric joint sealant used to waterproof the joint.

EXPANSION JOINTS

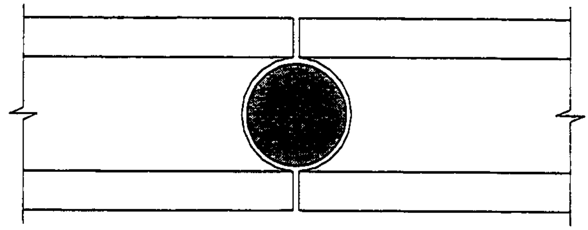
Special deeper column box units with special support:

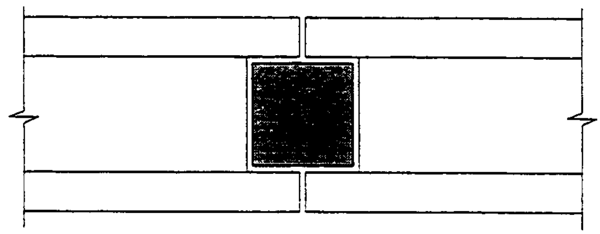
2 steel plates separated by a sliding plate to be inserted in the column box as support for the knife.



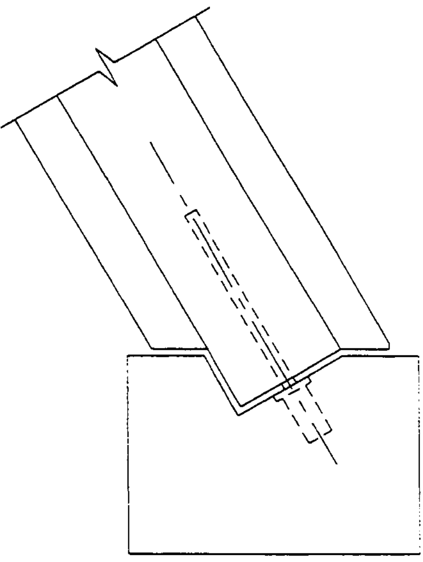
NOTE: Joint can be filled with a firesafe rock-wool and sealed with fire-resisting sealant and zinc coated for corrosion protection.

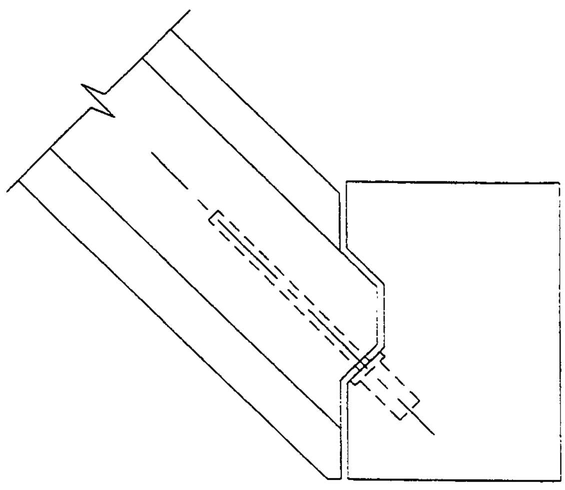
SPECIAL FRAMING FEATURES





INVERTED TEE BEAM AROUND COLUMN





SKEWED CONNECTIONS