A Prestressed Steel Space Frame

PAUL ROGERS

THE ST. JOHN BREBEUF Roman Catholic Church, Niles, Ill., completed in early 1966, reflects the contemporary trend toward circular houses of worship. The interior layout is arranged so that the priest and altar are virtually surrounded by the parishioners.

The building is 180 ft in diameter and has a 144-ft diameter flat conical dome topped by a 49-ft high tower and 26-ft high cross. Figure 1 shows a general view of the church. A uniformly distributed live load of 25 psf was used in the design.

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The architectural concept required that interior supporting columns be eliminated, yet the structure was required to support an imposing tower with stained glass windows.

FRAMING SYSTEM

To satisfy the architectural requirements, several conventional framing systems were tried but found to be unsatisfactory. Neither conventional steel framing, such as rigid frames or trusses, nor reinforced concrete con-

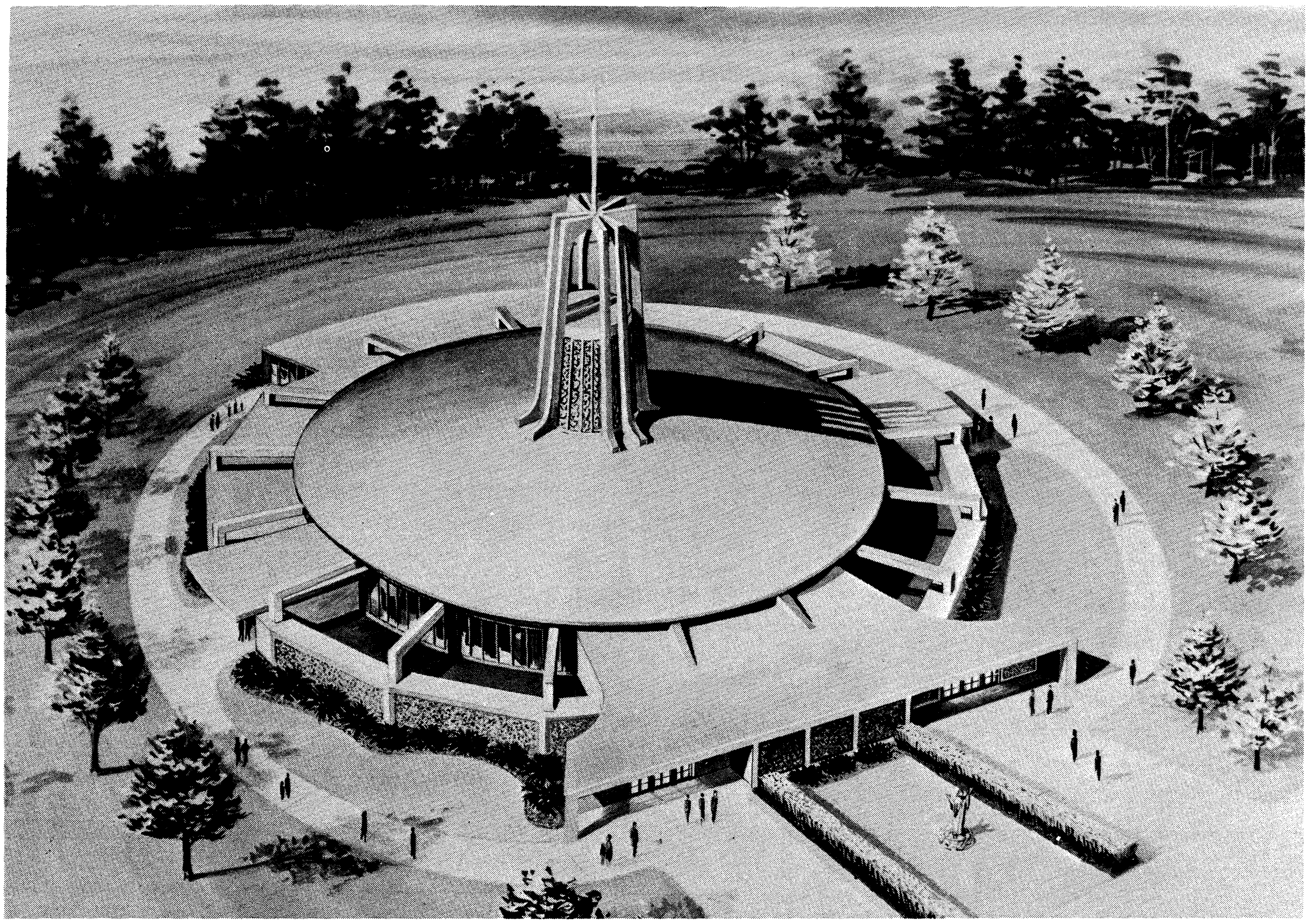

Fig. 1. St. John Brebeuf Roman Catholic Church, Niles, Ill.

struction provided a satisfactory solution. A prestressed, preloaded, steel space frame was found to provide a light, yet strong and economical dome structure suitable to the architectural design. The prestressed space frame adopted as the final solution incorporated a unique system of triangular space trusses (see Fig. 2). Each truss forming the dome is a shop welded pie-shaped wedge. Each triangular truss measures 14 ft-1$\frac{3}{8}$ in. at the widest point, and 2 ft-10$\frac{3}{8}$ in. at the narrowest. The truss depth varies from 2 ft-3$\frac{3}{4}$ in. to 6 ft-0 in. before sloping to 3 ft-10$\frac{3}{16}$ in. Roof pitch is 3:12.

Sixteen 14 WF columns, 16 ft high, support the dome at the periphery. The columns are connected by curved 18 WF beams which form a tension ring. At the top of the dome is a box girder compression ring, 30 ft in diameter. The eight legs of the 49-ft high tower rest on the compression ring. Figures 3 and 4 show details of the tower framing. All framing was A36 except for the tower, which was constructed of corrosion resistant A242 high strength steel plate.

After the dome was completely assembled, but before the tower and decking were placed, the structure was preloaded and the exterior steel tension ring was prestressed to create a simulated final loading with all the

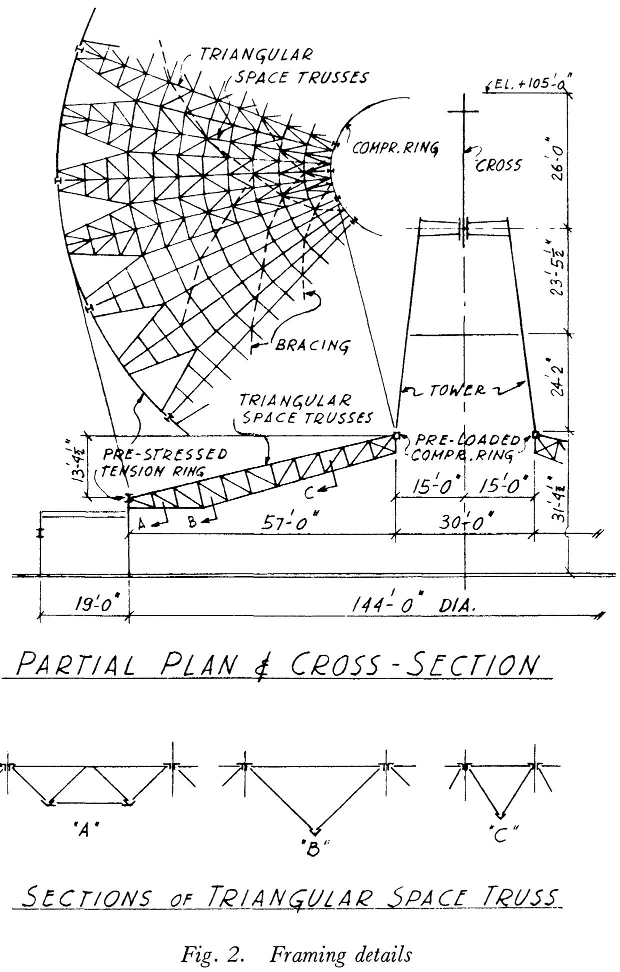

Fig. 2. Framing details

structural members fully stressed. The preloading was accomplished by suspending eight large oil cans (4 ft in diameter by 8 ft long), filled with sand of predetermined quantity, from the compression ring. As construction proceeded sand was let out of the cans so that at all times the structure and sand combined were equal to the total load.

The tension ring was post-tensioned by four cables, each composed of 28 $\frac{1}{4}$-in. diameter wire strands and having an ultimate load capacity of 216,000 psi. The cables were attached at four points around the ring, in order to reduce frictional losses. Figure 5 shows some of the prestressing details, and Fig. 6 illustrates the computations for post-tensioning the ring. Figures 7, 8 and 9 are schematic stressing diagrams.

DISCUSSION

An arch (or a dome) hinged at the supports and also hinged at the crown is a statically determinate three-hinged system. If the hinge at the crown is replaced by a 30-ft diameter ring, as in this framing, then the system may become an unstable four-hinged arch. To maintain stability in this structure, the following features were incorporated in the design:

1. Proper bracing was provided within the dome to prevent twisting movements about the crown. It was extremely important to provide adequate bracing within the surface of the dome in order to avoid twisting failure. Such twisting can occur about the apex of the dome and can be precipitated by any unsymmetrical loading of low or medium value. For erection purposes, such bracing members must be fabricated with "draw" to ensure tightness. Otherwise, the compression forces in the dome would create enough shortening in the main members to make the bracing loose and ineffective.
2. The central ring was provided with torsion resisting capacity. Under symmetrical gravity loading the 30-ft diameter ring takes only compression forces. However, wind acting on the tower and the cross, and the positive and negative pressures on the dome itself, depress one side of the ring while lifting the other, thus creating instability. Stability can be restored by torsional stiffness of the ring girder. This is the reason for the substantial box girder used in this framing.
3. Outward movements at the support were restrained, and the dome was preloaded so that incremental loads would not create additional deformations. Under gravity loading, the dome has a substantial outward horizontal movement. While the tension forces may be resisted by the exterior curved ring girders, the actual outward movement can not be prevented by conventional means. Therefore,

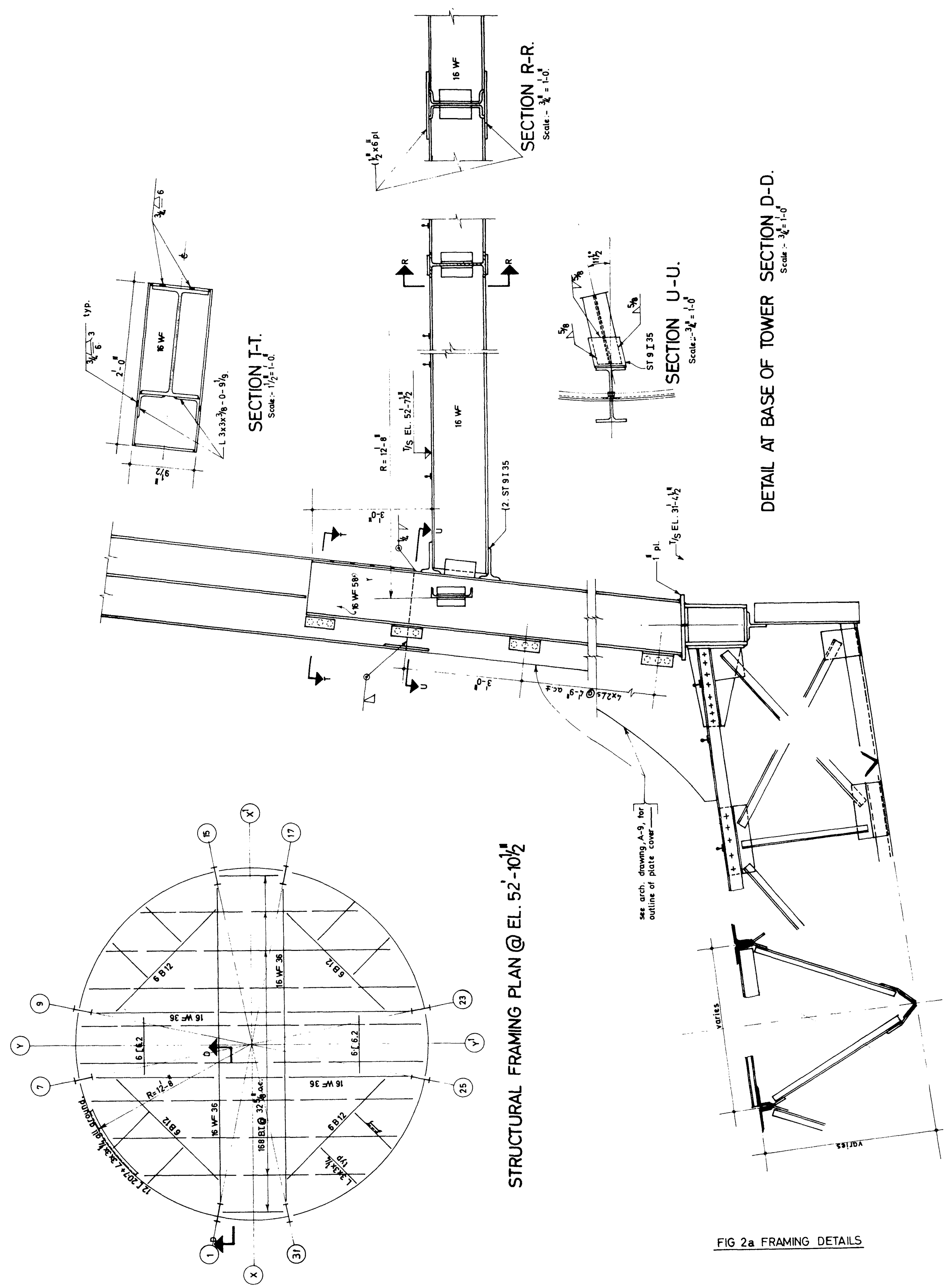

Fig. 3. Details at base of tower

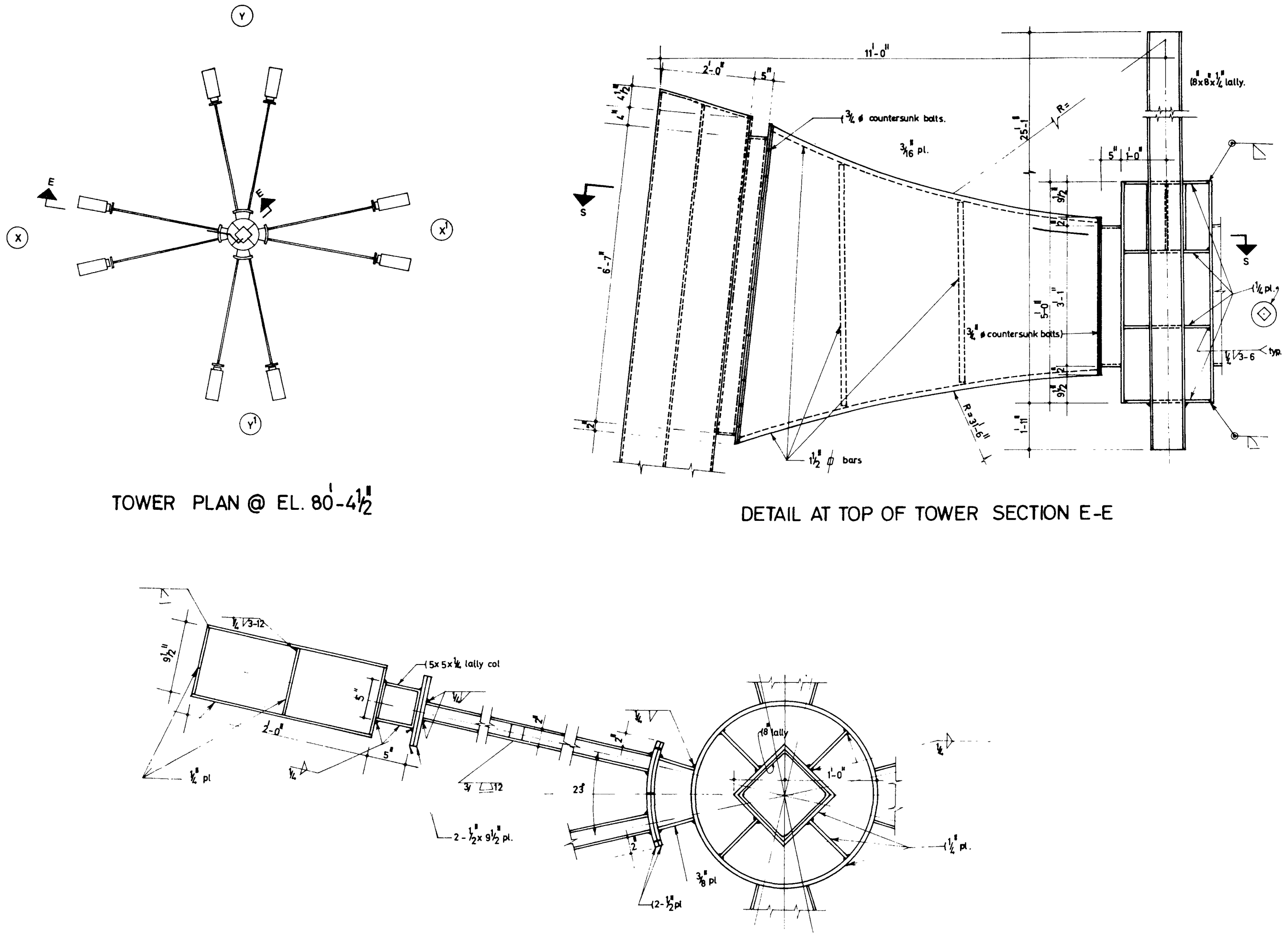

Fig. 4. Details at top of tower

prestressing was utilized. It was imperative, however, that prestressing should also be combined with preloading of the dome itself.

If prestressing were introduced without preloading, upward deformations (camber) would be introduced in the dome, since prestressing had to be applied before the tower and the decking were placed. Subsequent completion of the structure would counteract this camber, and cause additional movements in the dome. Simultaneous prestressing of the exterior tension ring and preloading of the compression ring created a simulated final loading with all the structural members fully stressed.

4. A light but rigid framing system was provided. (Cost of structural steel in place was \$5.50/sq ft.)

CREDITS

The architects for the St. John Brebeuf Roman Catholic Church were Gaul & Voosen, Chicago, Ill. Structural engineers were Paul Rogers & Associates, Chicago, Ill., now named Rogers-Cohen-Barreto-Marchertas.

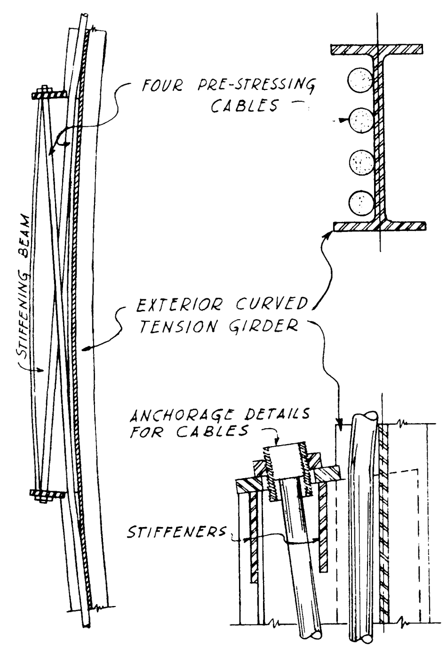

Fig. 5. Prestressing details

1. WIRE IS 0.25" DIA. ASTM-A421-58T (ACI 318-63)
2. MIN. ULT. STRENGTH GALV. WIRE — 216 ksi
3. MAX. FINAL STRESS = 0.6 ULT. — 129.6 ksi
4. MAX. INITIAL STRESS = 0.7 ULT. — 151.2 ksi
5. MAX. OVERSTRESS STRESS = 0.8 ULT — 172.8 ksi
6. RELAX. OF STRESS ≈ 4% OF AV. INITIAL — 3.1 ksi
7. LOSSES DUE TO FRICTION = $T_0 - T_x$
where $T_0 = T_x(e^{Kx+ma})$
$k = .0015$ per foot
$m = 0.250$ per radian
8. MODULUS OF ELASTICITY OF STEEL — 29.5 ksi
9. AREA OF WIRE — .04909

SAMPLE COMPUTATION

Tendon "A" - 28 wires, stressed from both ends, trajectory length 235'-11", total angular deviation: 180°

For critical stress point (45° from ℄ of anchorage)

$K_x = (.0015)(56.6) = 0.0839$, $m_a = .25 \times 45 \times \frac{1}{57.3} = .1963$

$e^{(kx+ma)} = 2.718^{(.2812)}$ — 1.325

Final Force (given) — 100 kips

Final Stress = 100 ÷ (28)(.04909) — 72.8 ksi

Initial Stress = 72.8 + 3.1 — 75.9 ksi

Initial Force = (75.9)(28)(.04909) — 104 kips

Overstress Stress = (75.9)(1.325) — 100 ksi

Overstress Force = (100)(28)(.04909) — 138 kips

For min. stress pt. 90° from ℄ of anchorage:

$kx = (.0015)(117.9) = .1769$, $ma = (.25)(90)(1/57.3) = .3926$

$e^{(kx+ma)} = 2.718^{(.5695)}$ — 1.767

Initial stress = 100 ÷ 1.767 — 56.6 ksi

Initial Force = (56.6)(28)(.04909) — 77.8 kips

Final Stress = (56.6) − (3.1) = — 53.5 kips

Final Force = (53.5)(28)(.04909) = — 73.5 kips

Av. Stress during overstress = ½(56.6 + 100) = — 78.3 ksi

Av. Strain during overstress = 78.3 ÷ 28,500 = — .00275 in/in.

Elong. during overstress = (.00275)(12)(117.9) = — 3⅞" ea. end.

Stress Loss due to relaxation of wire = (78.3)(0.04) — 3130 psi

FINAL		STRESS	FORCE
Tendon A	at critical point	72.8^k	100^k
	at minimum point	53.5^k	73.5^k
B thru H	at maximum-(anchor)	96.9^k	133^k

Note: The friction coefficients used are high for steel.

Fig. 6. Computations for post tensioning

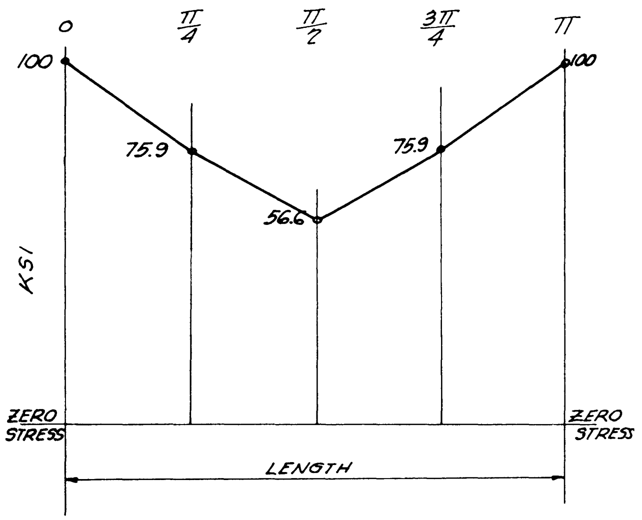

Fig. 7. Schematic stressing diagram for one tendon

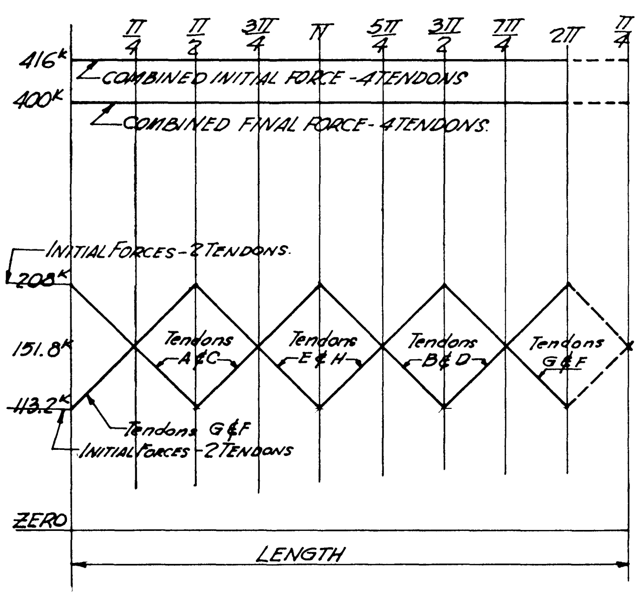

Fig. 8. Schematic diagram—combined forces

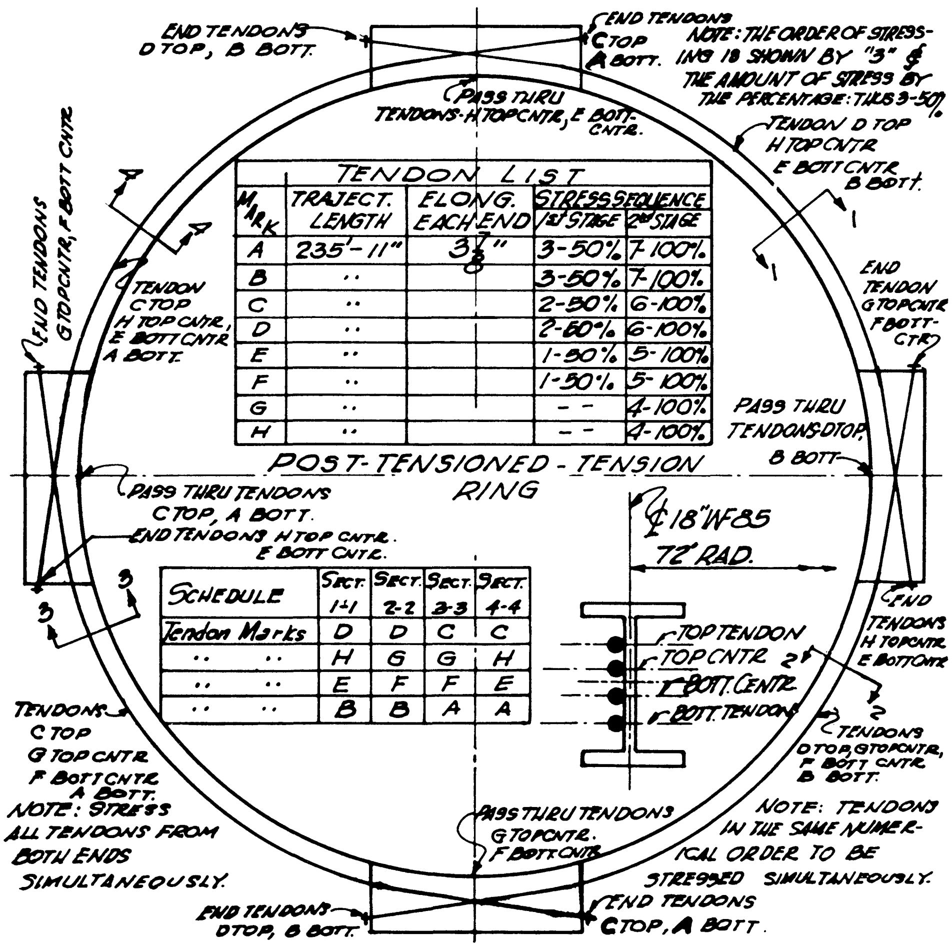
END TENDONS D TOP, B BOTT.
END TENDONS C TOP A BOTT.
NOTE: THE ORDER OF STRESSING IS SHOWN BY "3" & THE AMOUNT OF STRESS BY THE PERCENTAGE: THUS 3-50%
PASS THRU TENDONS-H TOP CNTR, E BOTT. CNTR.
TENDON D TOP H TOP CNTR E BOTT CNTR B BOTT.
END TENDONS G TOP CNTR, F BOTT CNTR
TENDON C TOP H TOP CNTR, E BOTT CNTR A BOTT.
END TENDON G TOP CNTR F BOTT-CTR
TENDON LIST
MARK
TRAJECT. LENGTH
ELONG. EACH END
STRESS SEQUENCE
1st STAGE
2nd STAGE
A 235'-11" 3 7/8" 3-50% 7-100%
B 3-50% 7-100%
C 2-50% 6-100%
D 2-50% 6-100%
E 1-50% 5-100%
F 1-50% 5-100%
G 4-100%
H 4-100%
PASS THRU TENDONS D TOP, B BOTT
POST-TENSIONED-TENSION RING
PASS THRU TENDONS C TOP, A BOTT.
END TENDONS H TOP CNTR. E BOTT CNTR.
℄ 18" WF85
72" RAD.
SCHEDULE
SECT. 1-1
SECT. 2-2
SECT. 3-3
SECT. 4-4
Tendon Marks D D C C
H G G H
E F F E
B B A A
END TENDONS H TOP CNTR E BOTT CNTR
TOP TENDON
TOP CNTR
BOTT. CENTR
BOTT. TENDON
TENDONS C TOP G TOP CNTR F BOTT CNTR A BOTT.
TENDONS D TOP, G TOP CNTR, F BOTT CNTR B BOTT.
NOTE: STRESS ALL TENDONS FROM BOTH ENDS SIMULTANEOUSLY.
PASS THRU TENDONS G TOP CNTR. F BOTT CNTR
NOTE: TENDONS IN THE SAME NUMERICAL ORDER TO BE STRESSED SIMULTANEOUSLY.
END TENDONS D TOP, B BOTT.
END TENDONS C TOP, A BOTT.

Figure 9