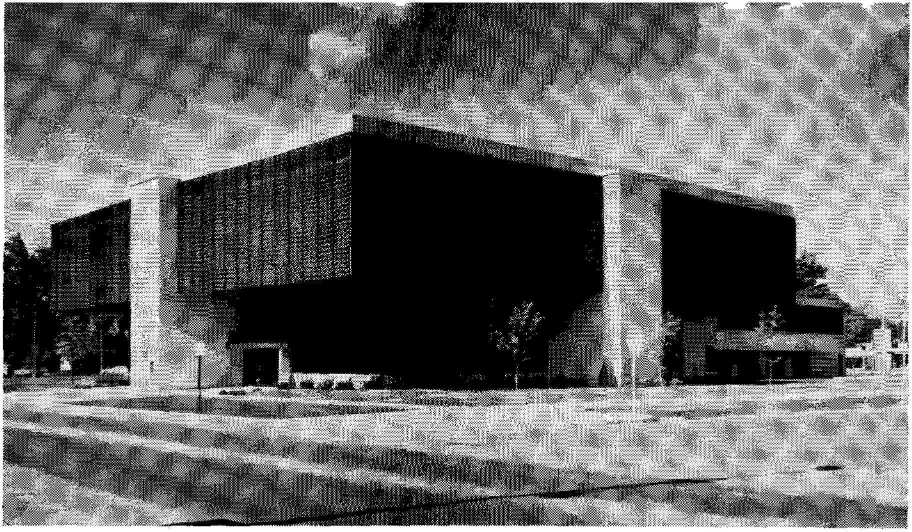

Poudre Valley National Bank—Ft. Collins, Colorado

30-In. Waffle Floor Slabs Cantilever 55 Ft.

by Jack D. Gillum*

INTRODUCTION

When the Poudre Valley National Bank of Ft. Collins, Colorado, decided to build new quarters for their banking facility, their purpose was to create a prestige building in a small community so unusually different and symbolic that the expression of the building would visually create a panorama of the banking operation and captivate the community.

To do this, the architect, James M. Hunter & Associates of Boulder, Colorado, decided on a column-free banking first floor area so that the entire banking operation would be visible to the public from all points of view.

Because the future plans of the bank dictated provision for expansion of the facility, it was decided to design the building for an additional seven floors thereby making the total eventual structure nine stories in height. The first phase of construction (see above) included the drive-

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in facility and the first two floors of the tower, with structural provision for the additional seven floors. A rendering of the complete nine-story structure is shown in Fig. 1.

The column-free first floor plan dictated the framing of the upper floors, and the four stair and elevator towers were therefore the sole vertical support for the structure. These towers were located midway along each side of the building (Fig. 2). The resulting plan dimensions were therefore—first floor, 96′-0″ x 96′-0″; second floor, 114′-2″ x 114′-2″; third to ninth floors, 119′-10″ x 119′-10″. It can be seen from Fig. 2 that the corners cantilever over 50 ft. for the second floor and 55 ft. for the third and future floors.

The structural problem was, therefore, to design the floors supported at the four towers located midway along each side. The problem was compounded to some extent on the second and third floors by the addition of the 32-ft. diameter opening in the floor, and by the fact that the first phase of the construction was up to and including the third floor, while the second phase would be built at some later date.

STRUCTURAL DESIGN PROBLEM

The resulting total structural design problem amounted to that of 1) designing the first floor, which was a conventional flat plate, with columns at approximately 20 ft. o.c.; 2) designing the second floor with the dimensions of 114′-2″ x 114′-2″; 3) designing the roof and future third floor with the dimensions of 119′-10″ x 119′-10″, 4) designing the four towers; and, of course, 5) designing the foundation, foundation walls and remaining miscellaneous portions of the building.

The deflection of the corners was, obviously, the primary problem and it was therefore immediately evident that a post-tensioned concrete floor was the only practical solution. Because the spandrels had to terminate

Fig. 1—Proposed Future Additions

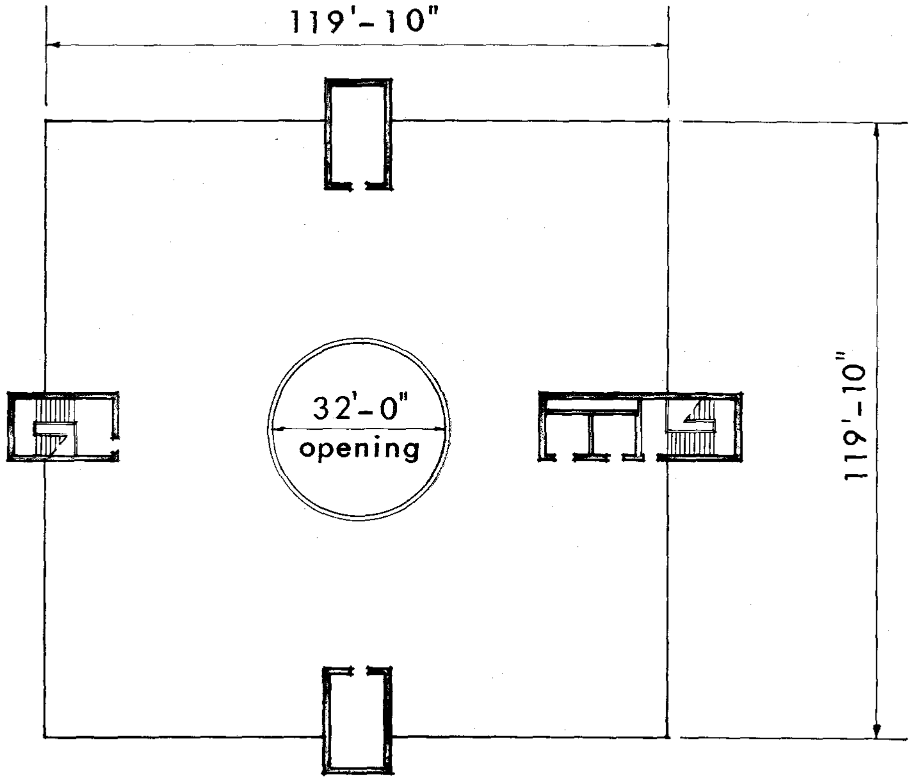

Fig. 2—Plan of Roof or Future Third Floor

at the tower, as can be seen from Fig. 2, it was decided to use a dia-grid waffle with a set of five main support girders spanning diagonally between the towers and with each corner cantilevering across these girders. This is schematically shown in Fig. 3.

To reduce dead load as much as possible, lightweight concrete with sand fines was specified. The design live load for the first floor was 100 lb. per sq. ft. and for the second and third floors, 50 lb. per sq. ft. plus 15 lb. per sq. ft. partition loads. Because the roof or future third floor was the largest of the two slabs to be designed, it was decided to design this slab first.

The first decision was whether or not to design the slab to withstand the second construction phase of the future floors. After estimating the concrete dead load of the future floors, it was decided to design the roof solely to support itself and design live loads, and that the future construction would have to be supported from a specially engineered set of simple and cantilevered forms. The design and arrangement of these forms were sufficiently considered at this time to assure that it could be done, though rather expensively.

The preliminary member sizes were determined by hand calculations and subsequently proved to be adequate and no changes were made at all in the final design. The third floor framing plan is shown in Fig. 4. The five main girders spanning between the towers are 48 in. o.c., 21 in. deep by 18 in. wide with a 9 in. slab, thereby giving a total depth of 30 in. The joists or beams are spaced at 96 in. o.c. and are 25½ in. deep by 16 in. wide with a 4½-in. slab and also a total depth of 30 in.

For analysis, the grid comprised 615 members and 362 joints. In actuality the constructed floor had somewhat fewer members, as the slab was thickened to 30 in. around the periphery of the circular hole and adjacent to the supporting towers.

DESIGN

Because deflection prediction and control was of utmost importance, an exact, or at least the most exact method available had to be used in the analysis. The method used was the unknown deformations or stiffness method. The solution of approximately 1000 unknown deformations, as is indicated by the number of joints and members, would be impractical by longhand, so the problem was set up and solved using the STRESS program written by MIT. The running time for an IBM 7094 computer to set up and solve for all the unknown deformations, moments, shears and deflections for one loading condition was 35 minutes.

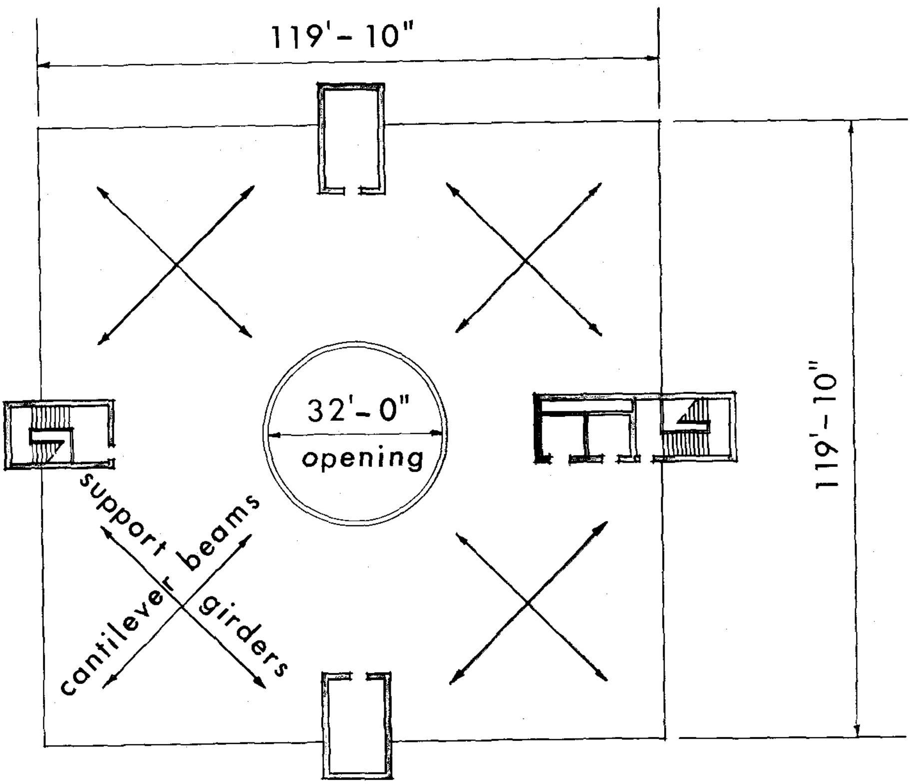

Fig. 3—Schematic Framing Plan

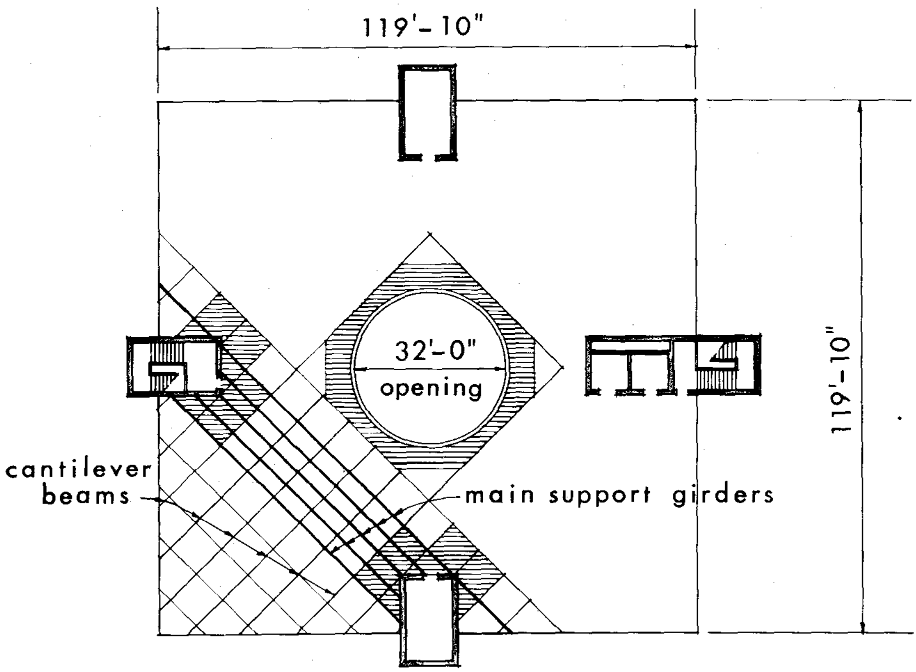

Fig. 4—Detailed Framing Plan

The initial run was for two loading conditions: loading 1 was for the slab dead load plus a 15 psf partition load, and loading 2 was for the ultimate total load. From these results moment and deflection curves were drawn for each member.

Fig. 5 shows the moment diagram for the cantilever members. Upon examination of these curves it was apparent, for economic reasons, to design the prestress for only a portion of the dead load. The preliminary cantilever tendon sizes were, therefore, selected on the basis of ultimate moments using an f_{su}-value for a bonded tendon of approximately 220 ksi. The final prestress forces in the cantilever beams range from 245 kips to 318 kips.

Fig. 6 shows the dead load moment diagram typical of the main girders spanning between the towers. Because this moment diagram is rather erratic it was decided to pick up the maximum shear which occurred at the joints indicated at each end of the girder. The tendon was then draped in a parabolic shape to pick up a uniform load matching the summation of these shears.

The design prestress force, P_f, for the first of these five girders was 580 kips and varied for the other four girders from 190 to 314 kips. The final computer run was then set up for dead load plus the effects of the design prestress force and for ultimate total load plus the effects of the design prestress force.

Fig. 7 indicates how the addition of the prestress force flattened out the dead load moment curves for both the beams and the girders. The

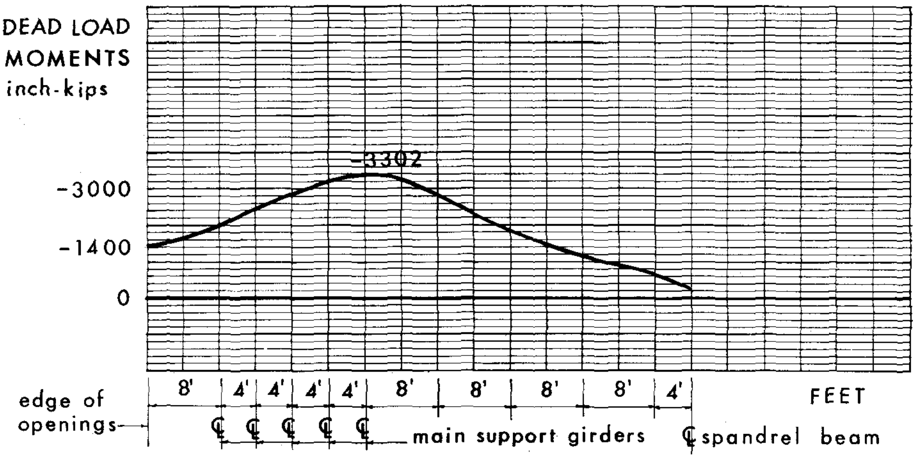

Fig. 5—Dead Load Moment Diagram—Cantilever Beam

dotted line is the dead load moment curve without prestress. The most important result of the prestress force was that the calculated corner deflection of 3.2 in. for dead load was reduced to 1.2 in. with the addition of the prestress. This was an acceptable deflection since a camber of 2 to 3 in. could be built into the slab and still have it behave satisfactorily.

The ultimate moments were then checked on the basis of the results of the total ultimate load plus P_f stress. Final ultimate moments were increased by the effect of the prestress

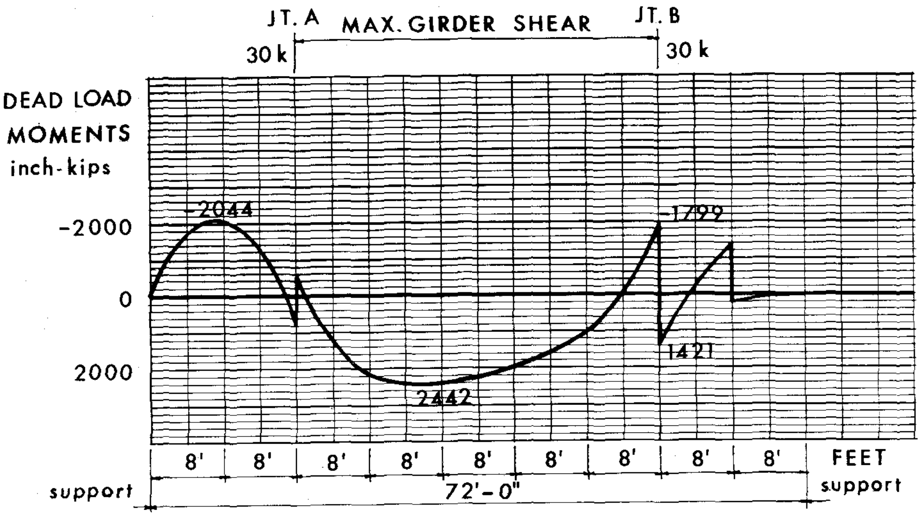

Fig. 6—Dead Load Moment Diagram—Main Support Girder

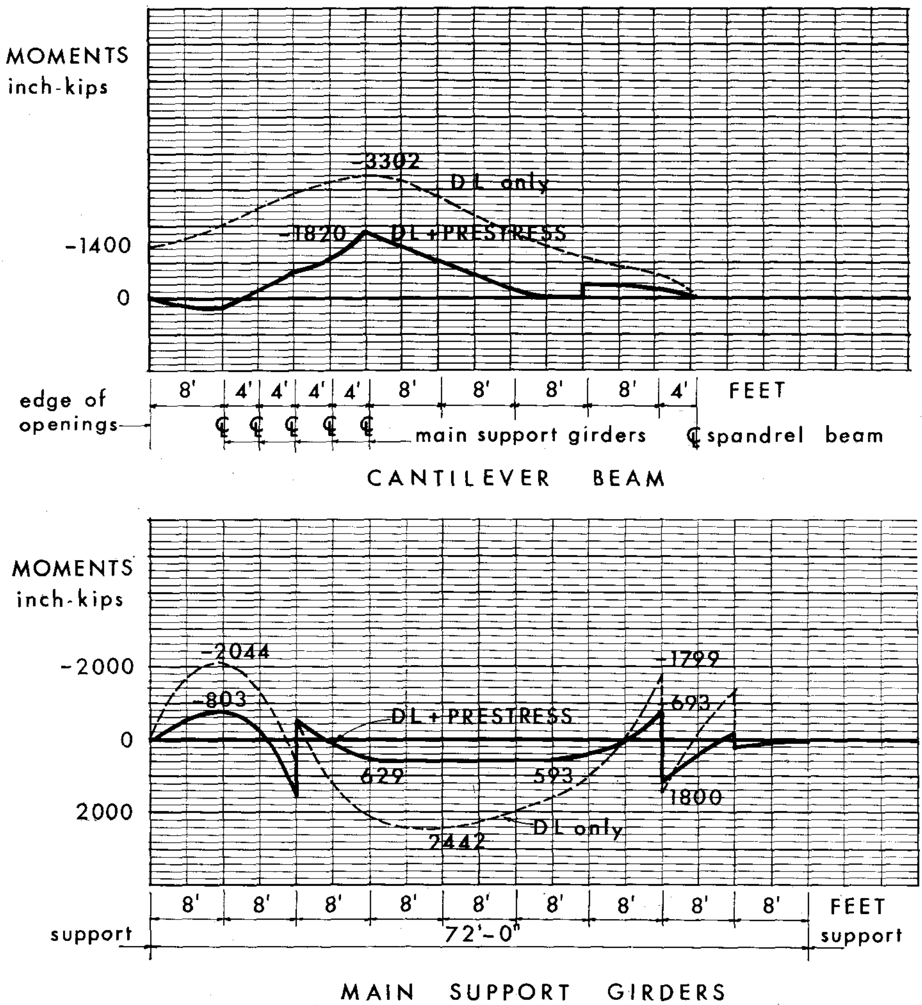

Fig. 7—Dead Load Plus Prestressed Moment Diagrams

force multiplied by its eccentricity in the beam, and the final steel was designed on this basis.

One important stress as shown by the computer was that of torsion which reached magnitudes up to 24,000 in.-kips for total ultimate load without prestress and was reduced to 3000 in.-kips for total ultimate load with the addition of prestress.

Using the current ACI formulas for shear, a computer program was written for stirrup design which combined the torsional stresses and shear stresses and designed and spaced the stirrups.

CAMBER AND DEFLECTIONS

The accurate determination of the long term creep and other prestress losses became an impossible task. After considerable research it was

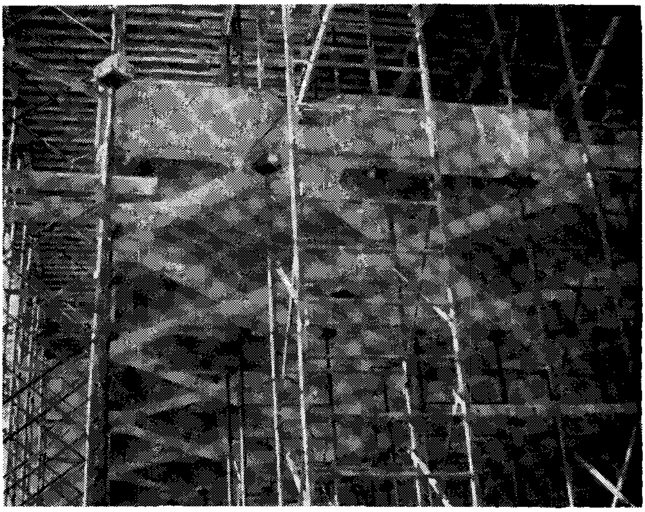

Fig. 8—Construction View Showing Underside of a Slab Corner

decided that the number of variables were too great and an estimated 2.2 for the long time deflection factor was used in the calculations. The corners were cambered by an amount 2.2 times the initial dead load deflection set forth by the computer. To further offset the prestress loss, it was specified that the tendons be re-stressed to their original force three months after the original stressing. This was done to assure that a portion of the creep and prestress loss deflection could be regained.

When the form work was removed from the second floor the corners deflected 1½ in. with a total variance in the corners of only ⅛ in. This compared to the predicted deflection of 1 in. The pre-camber of the second floor was 2 in., therefore the residual camber initially was ½ in. On the basis of the increased deflection of ½ in. over the predicted deflection, the camber of the third floor was increased from 2¼ in. to 3 in.

The building is now complete and occupied and it is interesting to note that the deflections of the corners conformed fairly well with the deflections calculated. The time-dependent deflections of the roof, however, were greater than that of the second floor and two struts at each corner had to be added to eliminate the differential deflection of the roof relative to the second floor. Deflection measurements to date indicate that the assumed long term multiplier of 2.2 was accurate. This is confirmed by the comparison of the calculated total initial and long-time deflection of 3.8 in. and the average of the measured deflections at the four corners of 3.97 in.

TOWER WALLS

After the design of the floor slabs was complete, the remaining portion of the building was rather simple. The tower walls are 14 in. thick to the underside of the second floor and 12 in. from there to the top. The concrete stresses are higher than acceptable for reinforced bearing wall construction so the vertical rebars were tied as in a reinforced concrete column and sleeved through the 30-in. portion of the slab to allow for movement during the stressing operation.

To ensure movement of the slab over the wall during prestressing, the bearing surface of the wall was steel troweled and a polyethylene bond breaker overlaid. The concrete design strength for the prestressed concrete slab was 5000 psi and for the towers, 4000 psi.

STRESSING

The problems and difficulties that occurred during the installation and stressing of the tendons were fairly typical for most post-tensioned construction. The congestion of the end anchorages and the stressing pockets on the second floor are indicated in Fig. 9; however particular attention was paid to the detailing of these pockets by the fabricator and, in general, the contractor had very little difficulty in the installation and stressing of the tendons.

The installation of the tendons in the second floor took approximately seven days and the stressing operation took five days. This was for a total of 114 tendons varying in size from 15 wires to 34 wires all sheathed in conduit.

The stressing sequence was to first

Fig. 9—Construction View Showing Congestion at Slab Edges

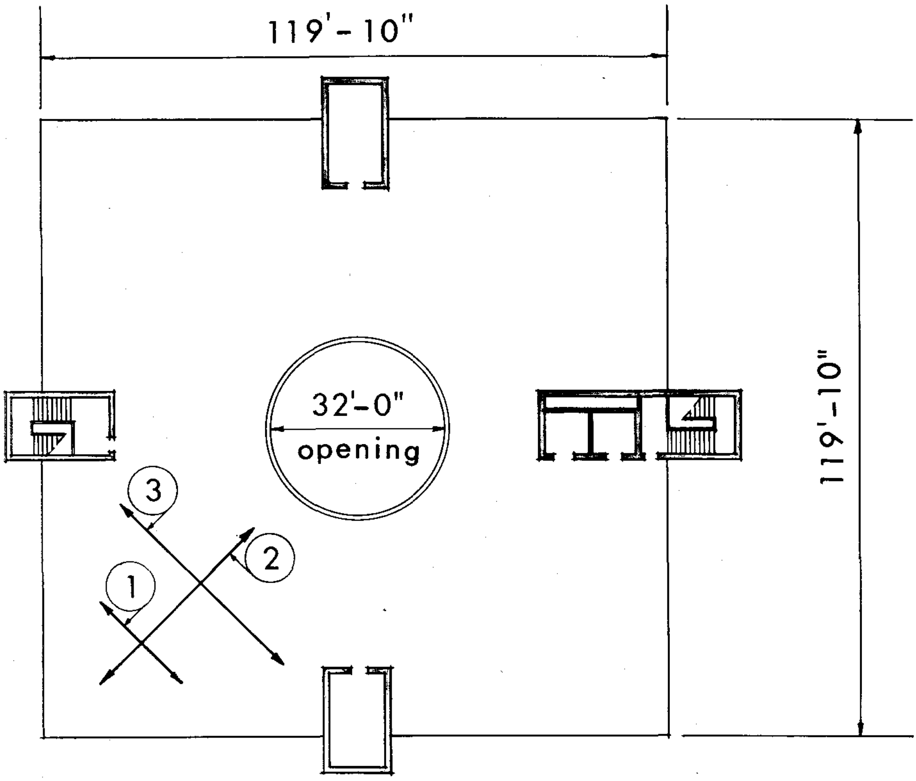

Fig. 10—Stressing Sequence

stress the diagonal tendons in the cantilevered portion of the slab (Fig. 10), then the cantilever beam tendons and finally the main support tendons.

To keep a close record of the camber or deflection of the slabs the contractor was required to record elevations at designated points along the periphery of the slab, along the corner to corner diagonals, and along the centerline of the main support beams. The readings were to be taken prior to stressing, after release of the forms and again in three months. The lift-off gauge readings were also taken to help determine the prestress loss during the three-month period after which the tendons would again be stressed.

CONSTRUCTION COST

The building was bid in February 1966 and construction began March 7, 1966. The price for the first phase of construction was $1,005,000 and the successful bidder was Frank Johnson Construction Company of Ft. Collins, Colorado. For the structural portion of the building, particularly the second and third floor slabs, the particular quantities and costs are as follows:

Forming—The dome pans were supplied by Jayhawk Fibreform Company and installed by the contractor for a cost per sq. ft. of slab of \$1.75 or a total cost of \$48,000.

Prestressing—52,000 lb. of grouted tendons were supplied by American Stress Wire Corporation at a cost of 42¢ per lb. or a cost of \$22,000. The placing and stressing was done by the contractor for \$8500, or a cost of 16¢ per lb. of tendon. The total cost per sq. ft. of slab therefore amounted to \$1.10.

Mild Steel—The re-bars, predominantly slab steel and stirrups, was ASTM A432 material and amounted to 52 tons. Fabrication and erection costs amounted to \$15,600, or a cost of 58¢ per sq. ft.

Concrete—The total amount of lightweight concrete for the two upper floors amounted to 1380 cu. yd. and was furnished in place at a cost of \$22.50 per cu. yd. for a total cost of \$71,000 and a cost per sq. ft. of \$1.15.

To summarize the unit costs of the second and third floor slabs:

Forming—\$1.75
Prestressing—\$1.10
Mild Steel—\$ .58
Concrete—\$1.15

for a total cost of \$4.58 per sq. ft. of floor slab.

CONCLUSION

In retrospect, if I were to make any changes in the design, I would more than likely use mastic-coated, unbonded tendons with only the ends grouted, primarily for ease of construction. The reduction in cost would also be a factor because very little prestress or mild steel would have to be added to satisfy the ultimate moment. I would also seriously consider the possibility of using a bottom-flanged shape in the cantilever section thereby reducing the prestress in those beams.

Finally, the use of the computer in the solution of problems such as this is an invaluable tool which frees the engineer from the chains of tedious hand calculations often impossible or impracticable in the solution of difficult structural problems.

Discussion of this paper is invited. Please forward your discussion to PCI Headquarters by January 1 to permit publication in the April 1968 issue of the PCI JOURNAL.