If the concave part were 1/10 in both directions, and if an initial even distribution of 50%/50% had been chosen, it could similarly be proven that the column strip/middle strip distribution would be 55%/45% in both directions. The width of the column strip, however, would only be 0.10 L wide and the column area would be a square within the same 0.10 L side dimension.

If the concave part of the cable had been made to occupy L/2 and the same 50%/50% initial distribution were assumed, the distribution between column and middle strip would become 75%/25%. The width of the column strip and the side dimensions of the column area, however, would now be 0.50 L.

In general, if we were to follow the typical flat plate distribution of reinforcing steel, it would result in tendons distributed throughout the plate in two directions. Their combination of high and low profile areas would result in a pattern as shown in Fig. 17.

Because of this and in order to prevent the need for threading of tendons, cumbersome and time-consuming placing schemes were required. If column offsets were part of the slab layout, additional complications resulted. Either the cable sequence at one column corresponded to a different sequence at the next column, or laterally curved cables were required. In either case, considerable difficulty arose both during planning as well as during placing of the tendons. Looking at Fig. 16, the following derivation is arrived at:

If brought to its ideal but impractical extreme, the concave part of the cable could be made equal to zero, the column-to-middle-strip distribution would become 50%/50%, but the width of the column strip and side dimensions of the column area would equal zero. Continuing the same line of thought but this time making the initial distribution 100%/0%, we arrive at a distribution between column strip and middle strip of 0%/100% in one direction and the reversed 100%/0% in the other direction. This, as it turns out, is the rather popular banded design. Its characteristics are shown in Fig. 18. The uniform distribution of the cables within the wide middle strip in one direction should be noted, and in the other direction the crowding of the 100% load into the narrow column strip. The banded prestressing layout yields a pattern where all column bands can be placed first, whereupon all uniformly-spaced tendons are placed afterwards. As to a stressing procedure, it should be recognized that all the uniformly-spaced tendons must be stressed first. If the bands were stressed first, an upward-acting overload could take place, sufficiently large to cause the partially-stressed slab to crack. A slight offset in the columns along the banded tendons would not drastically change the placing procedure. An interesting application of banded prestressed flat slab is the case of a slab with the columns placed in an equilateral triangular pattern (see Fig. 19). In this layout, three directions are of equal importance and, if mild steel were used, it would require the rather impractical use of three reinforcing-steel directions with the possibility in some areas of the need for three top layers and three bottom layers of rebars. (For slabs of some 5 inches in thickness, it is easily recognized that this would not be a very good solution.) In the case of banded post-tensioned steel, however, one direction may be picked to receive the banded tendons. Once stressed, this band would furnish a non-deflecting support line across which uniformly-spaced draped tendons may be run at 90° with the first selected direction.

As mentioned earlier, the column area deserves a special study. The negative moment is assumed to have a constant value within the column strip. The actual situation, however, is much more complicated. Reviewing the plan of a structural membrane shown on Fig. 20, we find that, under this theory, the area next to the column is acting as a funnel. Figs. 21 and 22 show how the analogous thin-shell surface is sharply sloping in this area. This agrees with Timoshenko's curve as shown on Fig. 12. Recognizing the importance of this many years ago, Prof. Nylander in Stockholm developed a circular symmetrical model as shown in Fig. 23. Within the slab thickness next to the column, it created a compression conoid. Without going further into the tests carried out by Prof. Nylander, it suffices to mention that for flat plate work, this is where the plate normally fails when subjected to failure loading, and this is the area that demands the designer's special attention.

Structural Design

Flat plates must be designed as continuous structural members. Unlike the mild steel-reinforced plate which may be designed by the empirical method or the equivalent frame method, the post-tensioned plate must be designed using the equivalent frame method as outlined in the ACI-318 Building Code, or any other method recognizing the elastic properties of the plate. Its design is generally performed in each of its main directions using a slab element, the width of which is taken from one center line of the bay to the next and using the corresponding columns. These often are assumed fixed at the floors above and below the slab. As with all prestressed concrete design, both service conditions as well as ultimate load conditions must be investigated. Because the draped tendons will create uplift in certain spans, notably in short ones, the necessity of alternating loading patterns has increased in importance. Also, with the thinner slabs, the resulting higher shear stresses at the columns warrant a more careful determination of the slab shears.

Continous prestressed-concrete structural design has another additional complication to consider. Disregarding the case of concordant cable profiles, the eccentricity of the tendons will cause bending of the concrete slabs which, if free to move, would deflect away from the supports. As this is not possible, secondary vertical reactions will develop at the supports sufficient to maintain the slab alignment at these points. The presence of secondary moments is best explained by the following illustration (see Fig. 24).

This phenomenon may be dealt with in a number of different ways. The most common approach is to calculate the moments resulting from the secondary support reactions. These are referred to as the "Secondary Moments" and are added to the bending moments caused by the dead and live load, and by the eccentricity of the prestressing tendons. Some confusion develops when factored loads and moments are calculated as to whether or not secondary moments should also be factored. Because these moments are the result of the resisting force from the prestressing cables, it is evident that this is not necessary. A second approach is "The Load Balancing Method." This has been used by many engineers and presents a simple way of arriving directly at the effect of the prestressing tendons without the cumbersome evaluation of the primary and secondary moments. Again, the difficulty arises when the calculations are extended beyond the balanced condition to evaluate other load cases including the ultimate load requirements. A third, and probably the most direct and logical approach is the determination of the "Effective Prestress Eccentricity" for the evaluation of the stresses developed during the service phase of design. This is done by determining a factor "P" so that E effective = P x E actual (see formulas and example). The advantage of this approach is that the exterior loads and the interior resistive elements are dealt with as entirely separate entities.

The fact that it is possible to drape the post-tensioning cables in such a way that little or no deflection takes place in the slab, with the exception of the small column area, means that the prestressed slabs are much less troubled with annoying deflections. Similarly, the large bending moments in the exterior columns often found in mild steel-reinforced slab and column frames are greatly reduced due to the same load-balancing effect. (Less deflection of the slab means less bending in the column.) Furthermore, due to the axial compression of the slab because of the prestressing in the tendons f_{pc}, an increase of 0.3 f_{pc} $b_o d$ + V_p in the available ultimate shears has resulted. However, the compressive stress f_{pc} to be used when establishing this allowable shear must be the average prestressing force and not the stress theoretically acting in the column strip. Furthermore, due to the concave curvature of the tendons near the support lines, a very conservative approach should be taken in determining the part V_p (the vertical component on the tendon force) that will help in carrying the dead and live load shears (See Fig. 10). Generally it may be stated, however, that whereas more elaborate calculations are required for post-tensioned slabs as compared with mild steel reinforcement, the post-tensioned structure is more forgiving in its basic behavior than the mild steel-reinforced structure.

A number of computer programs exist that allow an engineer to tackle the design of a continuous flat plate with scarcely more time expenditure than that used for mild steel-reinforced slabs. Most prestressing suppliers are capable of furnishing preliminary (and/or final) slab calculations to the individual engineering firms. With today's computer availabilities, the additional analytical work is of relatively little consequence. The extra effort is generally absorbed within a standard engineering fee. The engineer, however, must be aware of some of the prevailing special features which, if ignored, could result in uneconomical designs. Due to the ultimate design requirements or because of the final tensile stress level, the Code might require mild steel to be provided in addition to the prestressing cables. The ACI-318 Code requires the final tensile stress in mid-span to be limited to less than $2\sqrt{f'_c}$ in order to eliminate the need for mild steel in this region.

At the support, however, some bonded steel must always be provided. One reason for this is the relationship between the calculated bending moments to the actual moments (see Figs. 4 and 5). The actual moment shows a substantial peak value next to the column not properly reflected in the code-prescribed calculated moments. This bonded mild steel is furthermore required in order to develop the full punching shear of the plate at the column. This steel must lie within the shear cone next to the column. In prestressed design, the code further recommends that a minimum amount of compressive force is provided. The logic of this ties in with the fact that the allowable span-over-depth ratio has greatly increased compared to conventionally-reinforced concrete, and unless stressed to a certain minimum level, troublesome deflection could occur.

As ideal as the prestressing force is in resisting bending moments and preventing excess slab deflections, it also causes difficulties. The prestress causes the concrete to shorten; first elastically at the time of stressing, and later as longtime creep deflection, the latter taking place generally over the first 3 to 5 years of the life of the structure. This shortening must be considered both in the overall design and in the design of details. Where the frame can deform under this volume change such as in slender columns and in walls bending over their weaker axis, it is generally best to design this into the frame (see Fig. 25). Mainly at lower floors where this often is not possible, expansion details must be provided. This is often provided at the top of basement walls and in shear walls where they are subjected to bending over their strong axis. Especially rigid are basement wall corners (Fig. 26). The expansion details shown on Fig. 27 are not simple to provide. One is a sliding bearing of the slab on top of walls, another is sliding at the column pilaster. To take the thrust from the soil backfill, the joints need to be grouted later, thereby preventing these from adjusting to longtime creep in the slab. Where resting on columns or pilaster, the through-dowels must be sleeved to permit this movement. Again, they will freeze up when the subsequent pour is made above the slab (Fig. 28). Because of this, sometimes a slab is designed without expansion joints, and the resulting cracks are repaired later. This approach can often be defended from a cost standpoint.

Typical slab details have been shown in Fig. 29. These details are from a lift-slab construction project. The details, however, to a great extent would be the same for cast-in-place construction.

Post-Tensioned Design

Basic documents to be used in post-tensioned designs are: The ACI-318 Building Code Chapter 18, and "Recommendations for Concrete Members Prestressed with Unbonded Tendons" (ACI 423.3R-), reported by ACI-ASCE Committee 423, October 1, 1982.

Through the courtesy of the Post-Tensioning Institute, a reprint of a design example has been included--originally developed by the Post-Tensioning Institute and published in their book entitled, "Design of Post-Tensioned Slabs." This design example shows the various steps required to analyze a prestressed flat plate for an apartment building.

Lateral Load Design

Lateral load design for prestressed buildings is often handled in the same fashion as in the design of mild steel-reinforced frames. However, the required ductility for energy dissipation can be achieved by the appropriate combination of prestressed and nonprestressed reinforcement. A minor difference exists as the total dead load for post-tensioned buildings is less than conventionally-designed buildings. Also, because of the dead load of the slab being for a greater part balanced by the draped prestressing cables, in a case of a moment frame there is less negative bending moment at the columns to counteract positive wind load moments at these locations.

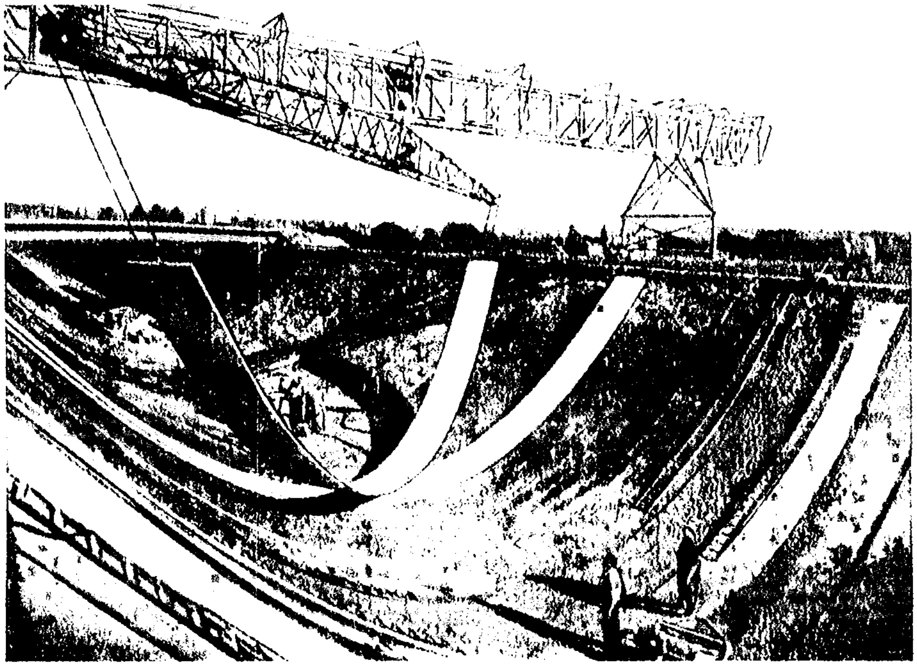


Fig. 1 Flexible Prestressed Concrete Liners

Fig. 2 Thin-Webbed Prestressed Concrete Girders

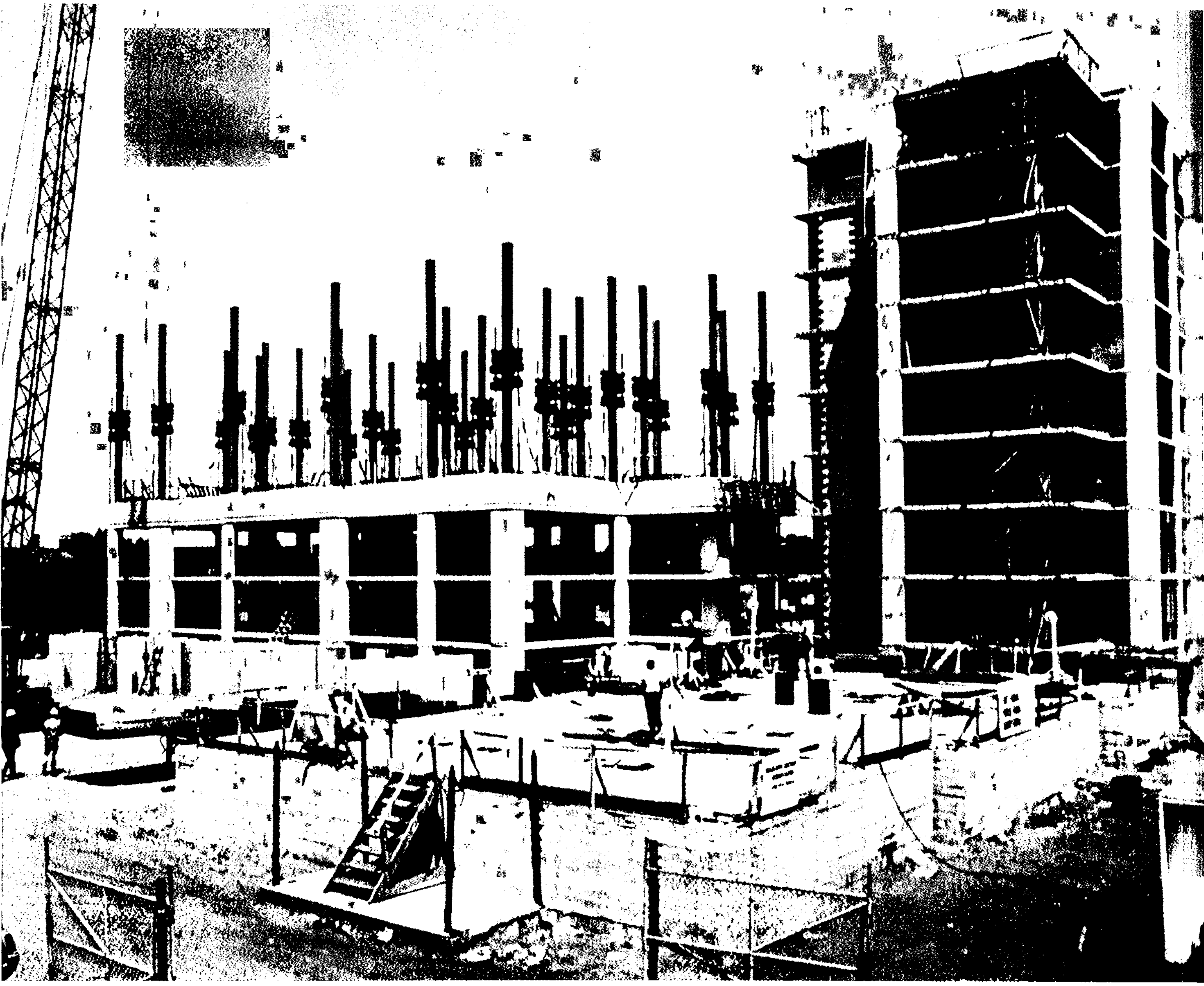


Fig. 3 A Typical Prestressed, Post-Tensioned Flat Plate Project Using the Multi-Leveling Component System

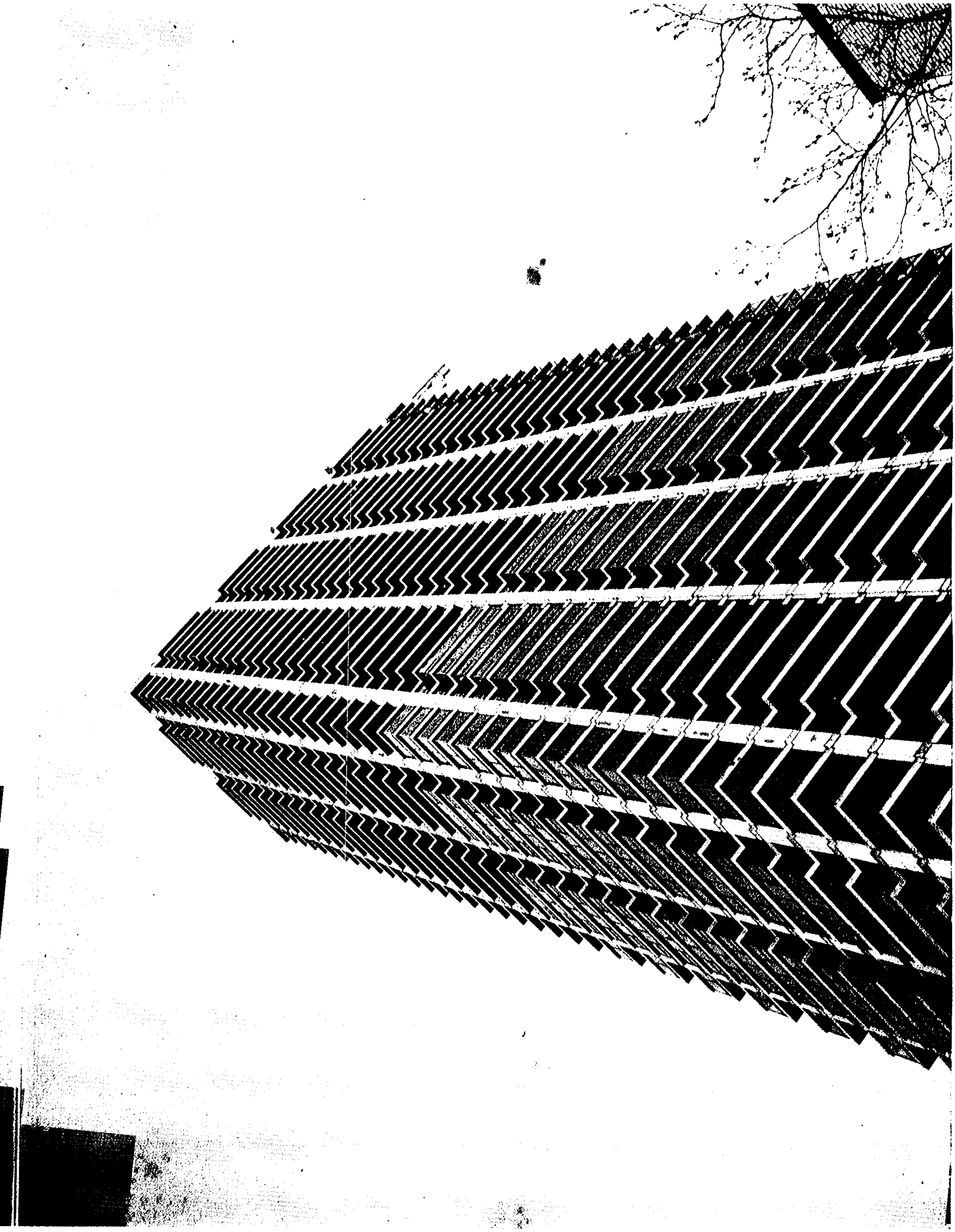
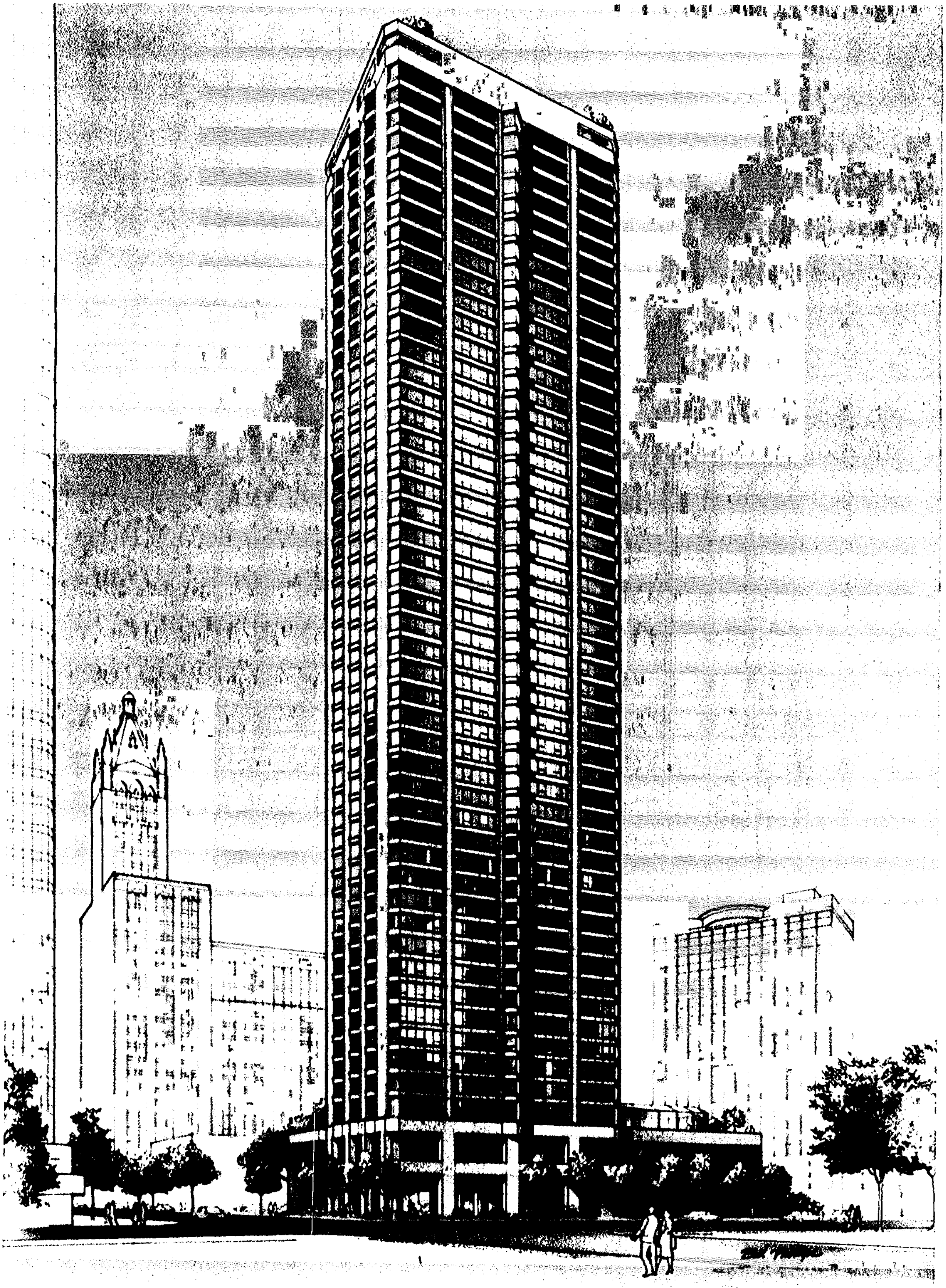


Fig. 4 56 Story Post-Tensioned Highrise (Huron Plaza, Chicago, Illinois)

Fig. 5 Huron Plaza, Chicago, Illinois



STREETERVILLE 400.

Fig. 6

50 Story Apartment Building
400 E. Ohio Street, Chicago, Illinois