Prestressing the CN Tower

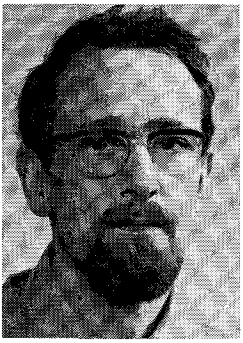
Franz Knoll, PE
Chief Engineer
Nicolet, Carrier, Dressel and Associates, Ltd.
Consulting Engineers
Montreal, Quebec

M. John Prosser, PE
General Manager
VSL Canada Ltd.
Post-Tensioning Subcontractors
Stoney Creek, Ontario

John Otter,* PE
Chief Engineer
Kilmer Van Nostrand Co., Ltd.
General Contractors
Downsview, Ontario

A major feature of this, the world's tallest structure, is that prestressing was used extensively in both the foundation and superstructure of the Tower.

This article describes the design and construction highlights in planning and building the structure especially as related to the post-tensioning operations.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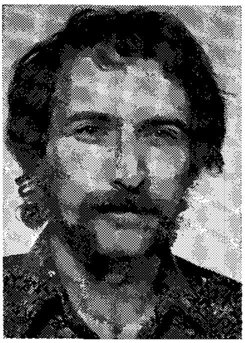
John Otter

Toronto's CN Tower is at 1815 ft the world's tallest† free-standing structure (see Fig. 1).

Built between 1973 and 1975, this giant communications and observation Tower is part of the Canadian National Railways Metro Centre redevelopment scheme which envelops 190 acres of railway land between the central business district of Toronto and the waterfront.

A major structural feature of the Tower is that both its foundation and superstructure (for most of its height) were prestressed using in-place post-tensioning.

The 1450-ft concrete shaft of the superstructure has a three-legged cross section which tapers upward. This central shaft was slipformed in about 8 months from June 1973 to February 1974.

In the summer of 1974 a skypod with seven floors was added to the superstructure to provide an observation restaurant and service facilities for broadcasters.

The basic supporting structure of the skypod is a set of twelve radially arranged cantilevered concrete walls which were initially tied into the slipformed shaft by means of stress bars. After the walls were completed, the cantilevered structure was stabilized laterally using a post-tensioned ring beam.

In March and April 1975, the concrete Tower was topped off by a 350-ft high steel mast. Erection was by helicopter. The mast will support various transmission equipment for television and radio.

As a precaution, the mast was subsequently clad in a cylindrical skin of glass-reinforced plastic to prevent ice

Fig. 1. Panoramic shot of CN Tower nearing completion.

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*Formerly, Project Engineer with VSL Canada Ltd., during construction of CN Tower.

†The Sears Tower in Chicago is 1450 ft and Ostankino Tower in Moscow is 1748 ft.

build-up during the severe Toronto cold season. There would be a danger of large chunks of ice breaking off and falling onto buildings and streets.

This paper will review the design considerations that went into constructing the Tower and, in particular, the prestressing methods. The various aspects that governed the design will be detailed as well as a discussion of the methods and procedures used for the post-tensioning operations. Especially, the prestressing of the foundation and shaft will be described including the auxiliary stressing of the anchorage zone and stressing of the ring beam.

Design Considerations

Because of the sheer magnitude of the Tower and the fact that fairly sophisticated construction methods were being applied on a rather large scale, existing technology was very carefully examined to establish its range of validity and practicality to new conditions.

The following were some of the major technical areas of concern especially the design and construction considerations relating to the prestressing operations. At this stage, no order of priority is implied since it was impossible to predict with certainty where the difficulties would be concentrated.

- The high magnitude of the forces to be dealt with implied the use of a large quantity of high strength steel tendons (a total in excess of 1000 tons was eventually used). This figure may not appear very sensational by today's standards; however, it should be noted that the majority of the force was applied basically to one single structural member, namely, the Tower shaft.
- At an early design stage, it appeared logical to construct the Tower shaft by the slipform technique. Since this method of construction is fairly sophisticated, it also meant that severe restrictions would be imposed on such items as time schedules and space requirements relating to detailing, placing, and stressing the tendons.
- Because of the record-breaking height of the Tower, it was anticipated that construction would last through at least one winter. Consequently, the concreting and post-tensioning operations would have to be planned to withstand very severe weather conditions. (In actuality, high winds and cold weather conditions caused many difficulties and construction delays.)
- The verticality and length of the tendons would be a significant departure from previously-known practice, and would introduce a whole new set of practical considerations, relating to the placing, securing, stressing, and grouting of the tendons. For instance, an accurate prediction of the post-tensioning losses was very difficult because of insufficient experience with this type of application.
- Grouting of vertical ducts would cause substantial hydrostatic pressure inside the ducts, the consequences of which were hard to predict.
- The decision to use post-tensioning for the foundation required extensive studies on the behavior of the (fractured) rock base subject to tendon stressing.
- Similarly, the decision to post-tension the ring beam at the supporting structure for the skypod posed intricate problems of geometry, relating to restricted space and the requirement to produce a uniform and symmetrical final stress in the twelve-sided polygon.
- The fact that part of the tendons had to be placed and stressed in the cold season, when the temperature of the concrete of the Tower walls would be below freezing, made it impossible to perform the grouting operation until many months later. Hence, the possibility of corrosion had to be considered.

The above items were only the major problems anticipated which would affect the post-tensioning operations. There were many more design and construction problems of a more local and immediate concern which came into existence during the construction period. Some of these will be discussed later on in the paper.

All the technical problems that could be anticipated had to be assessed and solved at the time of the design process. Unavoidably, there were many uncertainities and possible hazards arising from the unknown conditions and from the mode of construction.

This meant that many items had to be estimated and compensated for by built-in safety margins. This degree of conservatism was justified because of the importance and unusual proportions of the structure and the fact that even relatively minor construction problems and delays could not be tolerated from an economic viewpoint.

The authors were relieved and satisfied to find that the client basically concurred with the conservative decisions. In retrospect, time has shown that the built-in reserves were largely justified.

Because of the novelty of so many aspects in the design and construction of the CN Tower, all available technical and management skills had to be mobilized. The usually distinct separation of the responsibilities between the design team and the constructors had to be abandoned. On this project it became paramount that there be the utmost collaboration between the design engineers and the contractors.

For example, initial specifications and design plans had to be continuously adjusted and adapted to the conditions and difficulties encountered as construction progressed. The eventual success of this collaboration is manifested by the fact that this paper is a jointly authored report.

Prestressing the Foundation

The foundation of the CN Tower is based on the Ordovician rocks of the Dundas formation. The soils consist of horizontally layered shales and limestones, with occasional planes of clay-filled seams. Borings showed that random vertical cracks and separations also exist.

The footing slab, as it was eventually constructed, was the result of a lengthy design study. The decision to use prestressing, including the proportioning and layout of the post-tensioning tendons, was only one of the major features of the structure.

The geometry of the Y-shaped slab was largely determined by the shape of the Tower shaft, and by the fact that the foundation had to be situated at a relatively shallow depth below the future Tower base. Essentially, the foundation slab performs the same function as a conventional flat footing which supports the dead weight of the superstructure plus its own weight as well as overturning moments due to horizontal loads.

There exists, however, one significant difference from the usual type of flat footing besides the enormous scale, i.e., the spreading of the vertical loads to the interface of the rock base is, at least partially, performed by the Tower shaft itself, due to the tapering of the walls.

Thus, the function of the Tower footing proper is essentially reduced to that of the lower part of a flat footing, namely, that of resisting tensile stresses.

Obviously, in typical flat footings all these tensile stresses would be resisted by reinforcing steel or, perhaps, by post-tensioning in some cases. However, because of the unusual proportions of this structure, serious consideration was given to having the horizontal spread-

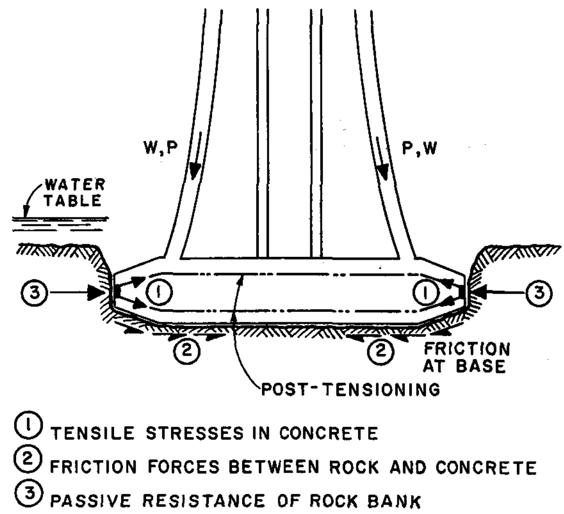

Fig. 2. Loading pattern of foundation slab.

ing forces resisted in another manner than by building in steel elements. The alternate principle would be analogous to the way an arched structure is supported, i.e., the rock base would provide the lateral resistance against movement of the springings. This method, of course, would provide a substantial savings in cost because the tension elements could be omitted.

In the case of the CN Tower, three mechanisms can be distinguished in which the spreading forces and resulting stresses would be dissipated (see Fig. 2):

1. By tensile stresses in the concrete of the foundation slab.
2. By friction activated at the interface of the rock and concrete.
3. By passive resistance of the rock wall around the perimeter.

The following reasoning was then used:

Because of the nature of the rock (thin layers with embedments of clay, and high pore water pressures in response to sudden dynamic loading), the mobilization of friction forces appeared doubtful and would occur only after some movement had taken place.

In turn, this mechanism might cause cracking in the foundation. This cracking would be most undesirable because the water table is located about 30 ft above the foundation underside.

Similar conclusions can be drawn for an edge resistance of the non-excavated rock. Reactive forces would be mobilized only after some elongation of the foundation slab because of the macroscopic compressibility of the rock (vertical cracks). Again, the foundation would be initially exposed to high tensile stresses that would result in undesirable cracks.

Thus, in order to safeguard against the entry of water, as well as to prevent large-scale cracking, it was decided to prestress the concrete footing. In this manner, the entire slab would be designed to be crack-free and carry no tensile stresses in the concrete.

During the stressing of the tendons it was confirmed that the foundation slab did indeed behave in the anticipated manner and that friction forces were

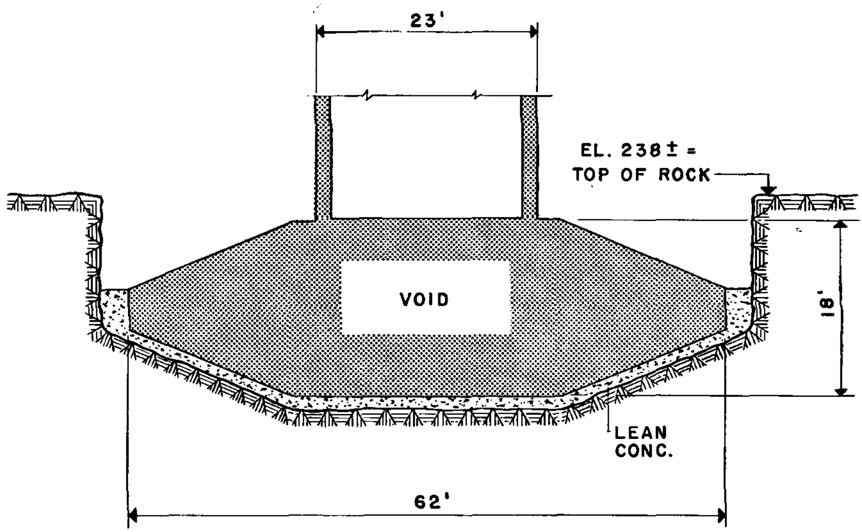

Fig. 3. Section through foundation.

not activated despite the fact that horizontal movements occurred.

The layout of the tendons followed a symmetrical triangular pattern. Furthermore, the hollow core of the footing slab had to be avoided because this space (see Fig. 3) was reserved for the stressing operation of the Tower shaft post-tensioning. The pattern eventually decided upon is shown in Fig. 4.

During construction, and particularly during the stressing sequence of the tendons, the effect of movements in the rock base had to be considered.

It was found that one of the critical failure mechanisms of the entire Tower structure would be governed by the concentration of shear stresses around the extreme edges on the leeward side of the footing. The relatively thin layers of the rock and its vertical separations indicated that indeed, this had to be a major consideration, since the risk of premature shearing of consecutive layers of rock had to be minimized.

Thus, any disturbance of the rock, because of the foundation moving on top of it during prestressing, had to be avoided in these zones. In particular, existing vertical separations in the rock could not be allowed to expand or form new fissures around the edges of the slab. This, in turn, meant that all movement of the slab at its extremities had to be eliminated and the point of zero motion, due to prestressing had to be near the end if each leg.

The above problem was solved by introducing open joints (see Fig. 4) near the center of the Tower, where flat jacks were applied to counteract shortening of the slab due to prestressing.

The stressing of the 48 cable tendons was performed by alternating with the pressurization of the movement controlling the flat jacks. The tendons were stressed from both ends to achieve a symmetrical pattern of stresses.

Each cable unit was initially stressed to 528 kips and then released to 400 kips. The average two-dimensional stress thus introduced in the slab is about 100 psi.

The movements that were observed during the stressing operation con-

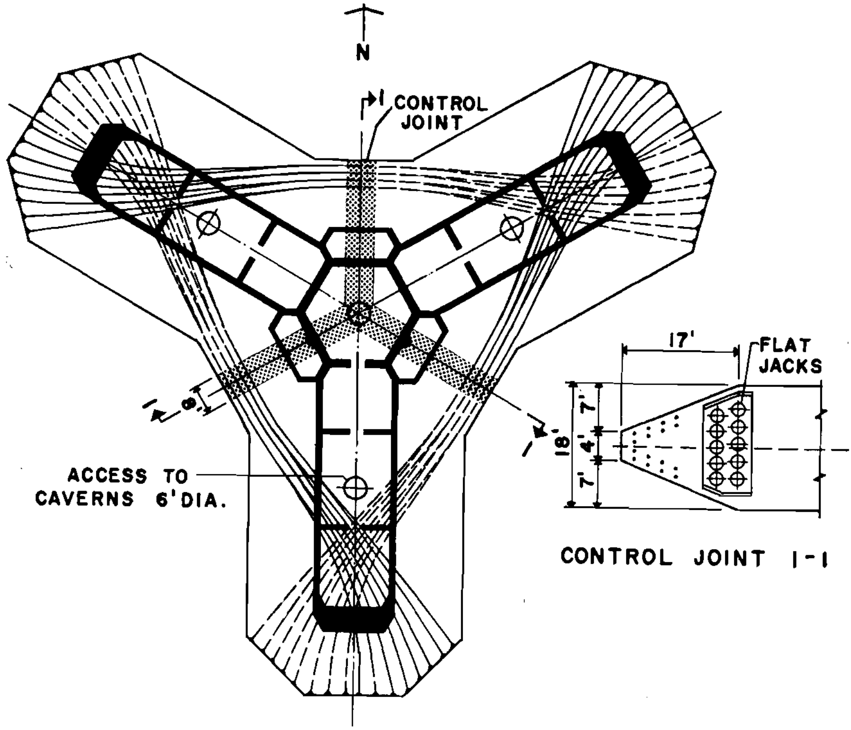

Fig. 4. Plan of foundation showing layout of prestressing tendons and section of control joint locating flat jacks.

firmed the anticipated behavior of the concrete slab. The points of zero movement were actually found to be located near the ends of the three legs.

An interesting question was whether or not the compressive stresses introduced by the post-tensioning would involve a portion of the rock base through friction. From the measured movement of the slabs it appeared that this was not the case. It was found that the actual displacements corresponded closely with the theoretical elastic deformations of the concrete. Therefore, it was concluded that all compressive stresses must have remained within the footing. Consequently, it was assumed that frictional engagement of the rock layers was largely nonexistent.

Fig. 4 is a foundation plan showing the tendon layout. Also shown is a section of the control joints and location of flat jacks. Fig. 5 shows the actual installation of tendons and mild steel reinforcement prior to concreting.

Problems and solutions

Some practical difficulties arose during the construction of the foundation.

The use of a semi-rigid sheath for the cable ducts proved to be troublesome in combination with the mass casting of a large concrete body. For example, many duct supports appeared to have been necessary. These were apparently not provided in sufficient number and the resulting displacements of cable ducts impaired the eventual sleeving-in of the strands. This operation was scheduled to take place after the concreting.

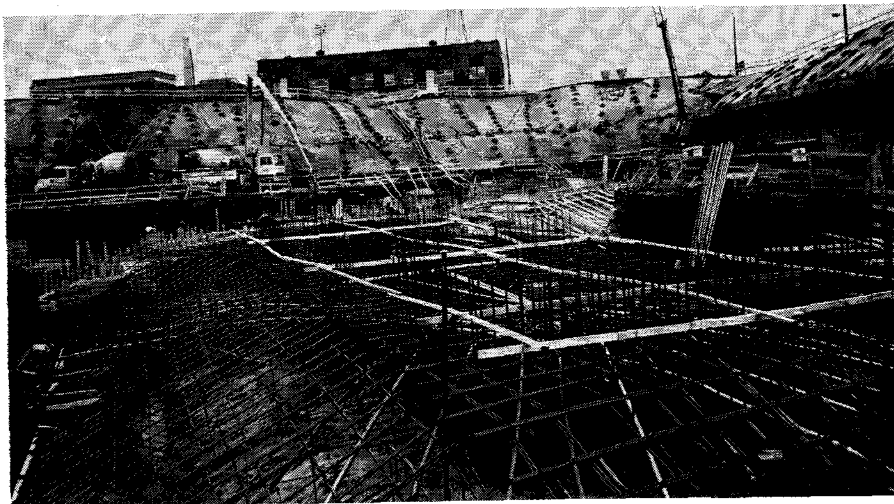

Fig. 5. Layout of prestressing tendons and mild steel reinforcement prior to concreting the foundation.

Unfortunately, the duct displacements resulted in delays and in the eventual loss of some of the tendons originally provided because of blocked or kinked ducts that would not allow sleeving-in of cables.

The alternating stressing from both ends also resulted in the loss of one cable, due to grip failure on strands in the far anchor head. This cable was later replaced but the energy contained in the tendon under stress proved to be so great as to balistically lift the stressing jack off its seat.

Fortunately, no injuries resulted from the incident. In retrospect, it is felt that a safety grip on the far end of the anchor might have prevented such accidents.

Prestressing the Tower Shaft

Design criteria

The basic design philosophy for the Tower shaft was established quite early in the study period. It was decided that the Tower shaft would be fully prestressed (post-tensioned). Hence, no tensile stresses under expected extreme loads were to occur in the concrete.

In this regard, extreme loads were defined to be maximum wind effects with a return period of 50 years. This description relates closely to the definition of wind loads in building codes which calls for a statistically based return period of 10 to 100 years.

For the extreme fiber in compression, a stress of 2500 psi was allowed as an upper limit on the gross area of cross section. This allowable stress includes all effects due to gravity loads, wind, and post-tensioning.

The walls were built with a specified compressive strength of concrete of 5000 psi. The percentage of mild steel reinforcement used in the vertical direction was between 0.5 and 1.0 percent.

The actual compressive strength of the concrete (from cyclinder tests) greatly exceeded the minimum specified concrete strength. However, at the time of design it was felt imprudent to specify very high strength concrete (and hence higher allowable stresses) because of the many uncertainties such as elastic shortening, shrinkage and creep, heat of hydration, and so forth.

In the event of extreme wind conditions (exceeding the values defined above), the Tower would actually become a "partially" prestressed structure with possible formation of cracks. However, this possibility was not judged sufficiently serious to warrant additional expenditures of steel and concrete, considering the rarity of this event.

Ultimate strength (limit state) analysis also showed that with the amount of post-tensioning and reinforcing steel furnished, a sufficient safety factor

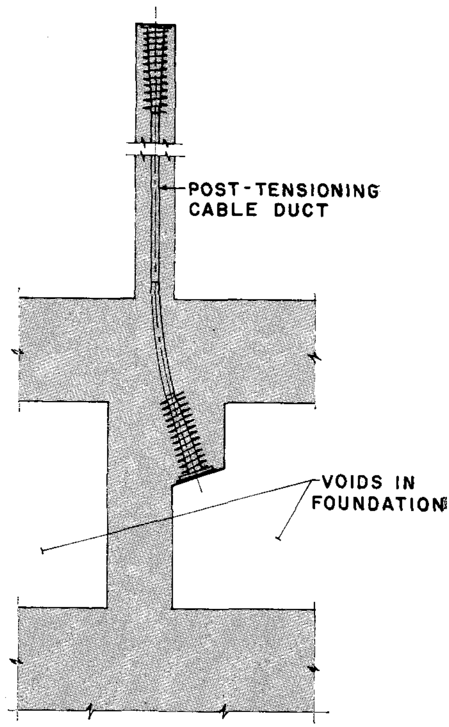

Fig. 6. Anchorage of Tower tendons.

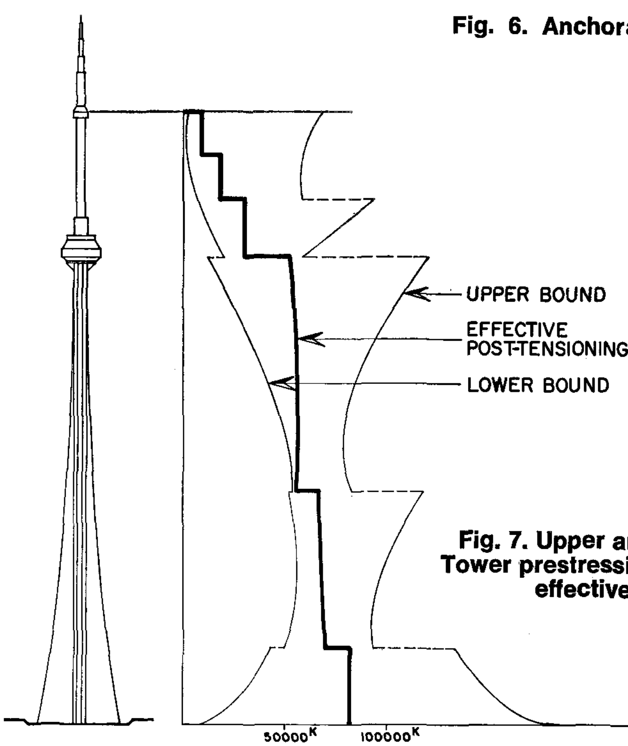

Fig. 7. Upper and lower bounds of Tower prestressing force with values effectively achieved.

against collapse was provided. (Note that the effective load factor for wind forces over the 50-year extreme period is approximately 2.3 for the limit state which includes all dynamic and second-order effects.)

The post-tensioning of the Tower shaft consists of 144 tendons which in turn are composed of 16 to 31 ½-in. diameter strands of seven wires each. The cable units are uniformly spread over much of the Tower walls, with the exception of the side faces of the wing sections.

All the Tower walls extend upwards with approximately constant thickness from the base to their top termination.

The location of tendons was governed by considerations of uniform stress and some practical problems which will be discussed below.

A concentration of all tendons at the extreme ends of the Tower wings would have slightly increased the ultimate strength of the shaft. However, this arrangement would have resulted in congestion with accompanying difficult geometrical and structural problems around the anchorage areas. Hence, a more uniform pattern of tendons was adopted.

All the cables were anchored in the base, with the anchor head accessible from the hollow core of the footing. The cables then extend to the top of the respective Tower walls with the top anchor head at the termination of the walls (see Fig. 6).

Stressing was successfully executed from the base with the exception of one cable group, the reasons for which will be discussed below. Stressing at the lower end was planned in order to minimize disruptions and delays to the slipform operation.

Note that the sheltered areas in the foundation area (see Fig. 6) allowed the intricate stressing operations to proceed while work overhead was continuing. However, when stressing was conducted from the top, as it had to be done for one group of cables, the operation caused a 2-week delay.

Fig. 7 shows the constraints placed upon the prestressing design in the Tower walls together with the effective amount of post-tensioning eventually achieved. (The actual distribution of tendon units throughout the Tower shaft is summarized in Fig. 8).

It may be noted from Fig. 7 that the most economical force diagram should follow closely the lower bound curve. In fact, the effective prestressing force does so only in a very approximate manner.

Apart from some considerations relating to construction stages, this approximation arises from the very practical problem of detail design, and of construction sequence. For example,

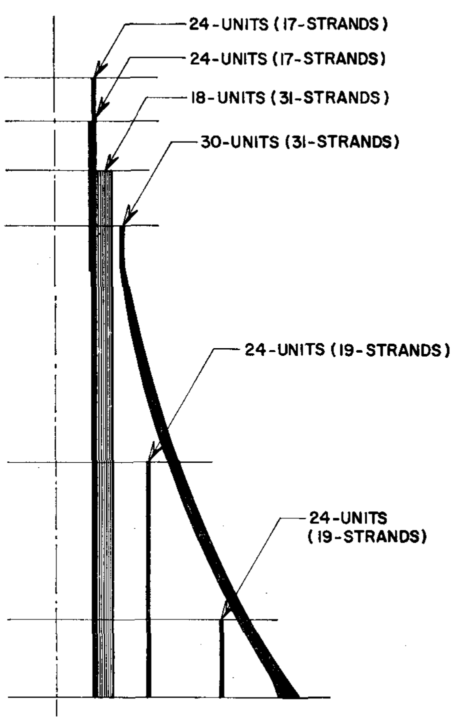

Fig. 8. Distribution of tendon units through Tower shaft.

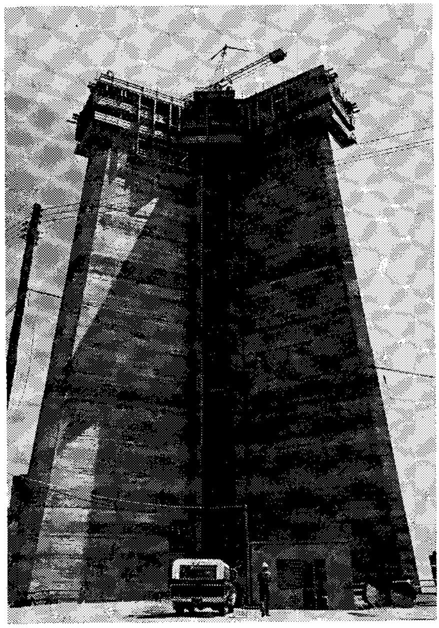

Fig. 9. Slipforming of Tower walls (August 1973).

the top termination of the tendons necessitates a halt in the continuous slipform operation. Rather bulky and delicate equipment had to be hauled in for placing the cables and anchors. The concrete casting, curing, and finishing operations as well as the reinforcing steel setting cannot proceed while the post-tensioning tendons are placed. That is, both operations cannot be conducted simultaneously.

Hence, the number of times the slipforming could be disrupted, including all preparations for stop and go, had to be kept to an absolute minimum.

It was therefore decided to sacrifice some materials in order to allow for a smoother slipform operation. To further expedite matters, the post-tensioning tendons were arranged in large groups and were conveniently placed so that all top anchorages of cables were at the top terminations of individual Tower walls.

This feature also avoided the difficulties that would have originated from a cable termination detail in the face of a wall.

Figs. 9 and 10 show various stages of slipforming the Tower shaft. This slipforming operation took about 8 months to complete.

Contrary to an earlier intention, no tendons were provided in the side faces of the Tower supporting wings. These are triangular in elevation and no top head of these walls exists. The changeover of cables from these walls into others would have posed too many difficulties for placing and, thus, had to be abandoned.

This change caused some rather large stress concentrations at the top of the cross walls. However, these stress concentrations were resisted using reinforcing steel and auxiliary stress bars in one instance (see p. 107).

All the cables were introduced in the walls in one piece, thus avoiding coupling devices and intermediate stressing stages. This produced rather large stress elongations (see Table 1).

It was also necessary to introduce some reinforcing steel for temporary construction conditions, with part of the Tower shaft unstressed and the slipform moving ahead. Some of this reinforcing steel would not have been needed in the final structure but its cost was considered a fair expense in exchange for greater ease of construction and continuity.

Table 1. Tendon elongations for various cable groups.

Cable group	Cable length, ft	Total elongation, in.
A1	1487	110
A2	1378	103
B	1262	98
D	562	44
E	180	15
FG	1125	89

At the time of design neither the exact sequence of construction nor the relationship of the construction process to the seasons were known. Weather conditions were anticipated to cause delays and difficulties but to what measure this would occur was impossible to assess during the planning stage.

Other anticipated difficulties included the blockage of cable ducts due to denting or chunks of wet concrete falling into the sheath. The possibility of cleaning the ducts before inserting the tendons was very doubtful.

In general, the placing and stressing of vertical post-tensioning units of this length could meet with many unforseen difficulties. It was quite probable, therefore, that individual strands, or entire units, would be lost. Recovery or replacement of these would cause excessive delays and costs.

In addition, much uncertainty existed during the design stage as to the prestress that would effectively be achieved at the critical points of the shaft. This was due to lack of experience in estimating the amount of prestress losses and due to practical difficulties which could not be predicted.

Therefore, a somewhat larger amount of prestressing than initially called for was introduced to compensate for this uncertainty. This excess of post-tensioning was continually reassessed and adjusted as the operation proceeded in relation to the hazards and difficulties anticipated for each group of stressing and the experience gained during construction.

Compensatory measures included the provision of additional ducts which, if not used, would be left empty and grouted simultaneously with the cables. Also the choice of diameter for the sheath used was made in such a way as to allow for additional strands to be placed over and above the projected number. This flexibility allowed the

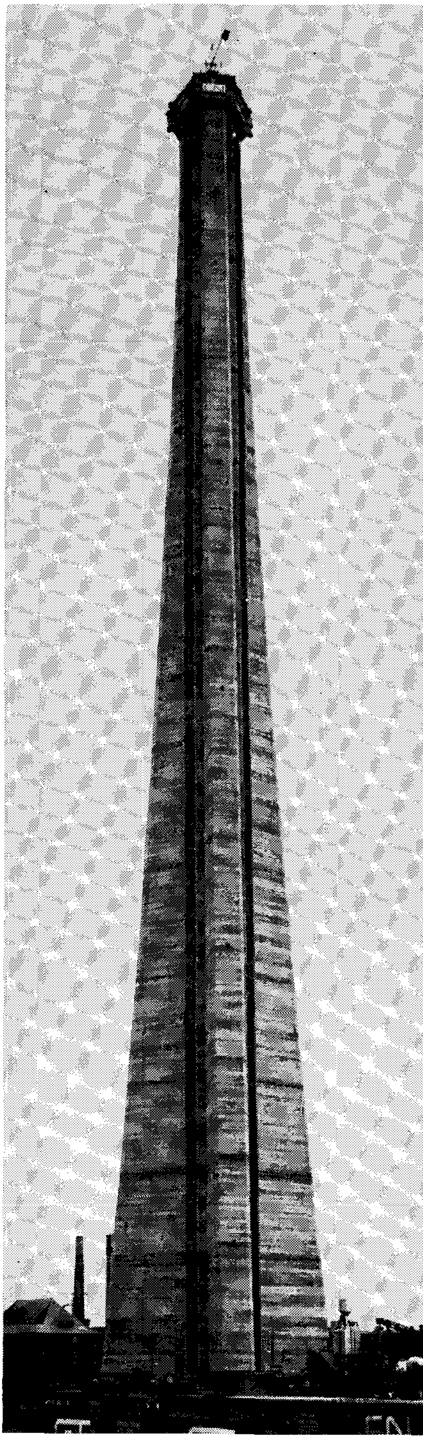

Fig. 10. Slipforming of Tower walls (November 1973).

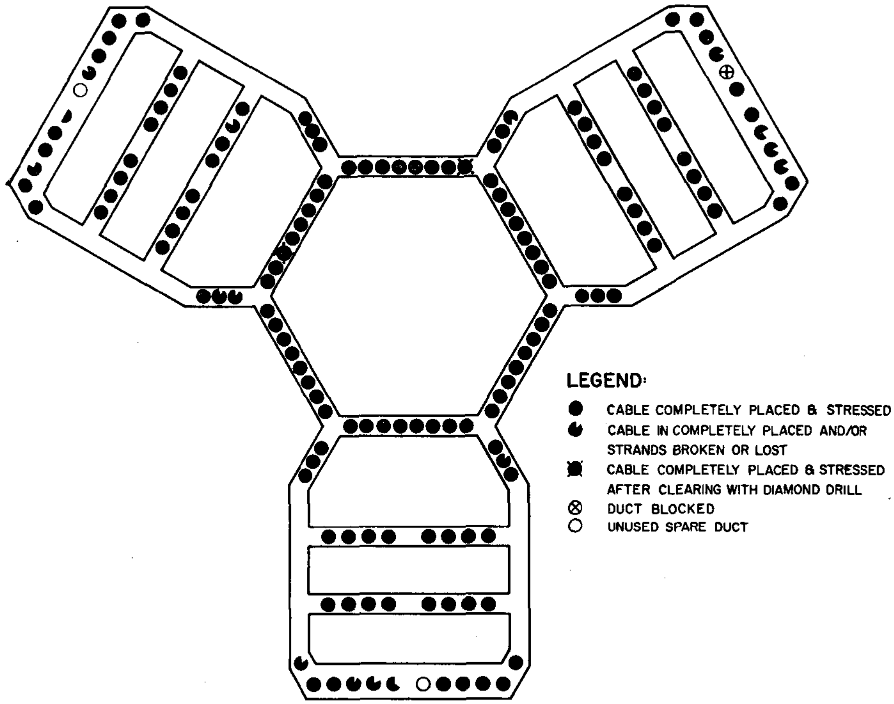

Fig. 11. Historical review of post-tensioning results.

possibility of replacing tendons that were partially or entirely lost or impossible to place or stress.

Fig. 11 illustrates the arrangement of these precautions and their final usage which largely justified their implementation.

The practical instances that led to this will be briefly reviewed below:

1. All ducts were built-in with diameters of ½ in. or somewhat larger than required for the projected amount of strands to be inserted. This precaution would have allowed an additional 20 percent of strands above the projected number.

2. For one group of cables, two spare ducts were inserted for every ten cables to be placed. In actuality, these ducts were used eventually since it proved impossible in some instances to remove blockages, or to insert all the strands required.

3. An overdesign of about 5 percent was provided in the number of strands to compensate for the uncertainties due to prestress losses and unforseen construction problems.

4. As a back-up measure, the top anchor head was chosen to be of the stress anchor type. This precaution allowed stressing to be conducted from the top end either as a replacement to base stressing or as a supplementary measure for friction and other losses in excess of that anticipated.

Fig. 11 summarizes pictorially the following items where things went wrong and could be compensated by the precautions provided:

1. Ducts blocked.

2. Loss of strands due to problems with tendon threading.

3. Strands lost due to loss of grip at the brake and at the stressing jack.

4. Strands snapped during stressing operation.

In some instances the built-in reserves were more than sufficient whereas in other cases they were used up almost completely.

Prestress losses

It is fair to say that an accurate prediction of prestress losses due to creep, shrinkage, elastic shortening, and steel relaxation is difficult to determine. It might also be said that very great accuracy is not really necessary. This proved to be the case with the CN Tower. However, it was necessary to investigate the losses problem to some extent before moving on with confidence.

Recent revisions to the AASHTO Specifications and PCI recommendations suggest that prestress losses have sometimes been grossly underestimated in the past. Coincidentally, the service record also suggests that these underestimations have not been critical. Furthermore, an overestimation of prestress losses does not necessarily produce a conservative design.

The evidence for underestimation was deduced from tests which appear to have accounted for all the significant parameters quite rigorously.

The parameters and their variations in relation to the CN Tower are described below. The basic approach taken was to compare the conditions in the Tower, item for item, with those existing in standard bridge construction for which experience is extensive.

1. *Concrete shrinkage*—The concrete in the Tower did not exhibit shrinkage characteristics that differ markedly from the norm. In addition, much of the concrete was bound to be many months old at the time of tensioning.

2. *Elastic shortening*—This would be quite variable from cable group to cable group and from level to level. Since much of the effective prestress is derived from the dead weight of the structure and because most of the weight would be acting before tensioning of the major cable groups (i.e., longer cables), it was judged that the condition in the Tower was superior to a standard application.

3. *Concrete creep*—The concrete in the Tower did not differ from "normal" concrete in its creep characteristics. Since the summation of prestress from post-tensioning and from the structure's dead weight was within the normal range, no particular departures were anticipated. Actually, the situation would be more favorable for the longer cables since much of the creep due to dead weight prestress would have dissipated before stressing.

4. *Strand relaxation*—Recent findings regarding relaxation losses were accepted as being qualitatively correct, i.e., that losses increase as the initial strand stress increases. It was, therefore, established that the losses to be expected in the Tower should not exceed those in normal practice. In fact, there was every expectation that they would be less. Furthermore, since much of the prestress effect was due to weight of the structure, variations in prestress loss would have an even smaller effect on performance than is normally the case.

The breakdown of anticipated losses for the longer cables was:

Shrinkage	2.5 ksi
Elastic shortening	1.8 ksi
Concrete creep	11.0 ksi
Steel relaxation	13.0 ksi
Total	28.3 ksi

The total loss was, in fact, so close to the traditional value of 25 ksi that it was decided to abandon the sophistication inherent in the extra 3.3 ksi and to adopt the standard value.

Friction and wobble losses

There were several factors in the post-tensioning of the Tower that were outside the realm of traditional experience. These included:

1. Cables that were extremely long (up to 1500 ft).

2. Ducts that were essentially straight and vertical for their full length. The only curves of significance in most of the ducts were those of the bottom anchorages.

3. The use of rigid sheathing. This was suggested by the prestresser, essentially as a means for increasing the reliability of the ducts but would also have a significant effect on friction.

4. The use of galvanized sheathing. This made the ducts entirely unrusted at the time of stressing.

5. The use of templates to set the ducts with minimum deviations.

6. The use of "oversized" ducts for provision of space capacity.

These conditions, if cumulated, produced uncertainties far in excess of the ones concerning other losses. Thus, a careful procedure had to be used, involving continuous application of experience gathered during the construction, and constant updating of procedures from newly gathered information.

It should be noted that the manufacturer's guaranteed cable coefficients of wobble and friction were $K = 0.0005$ and $\mu = 0.2$, respectively. This meant that 920 tons of vertical post-tensioning would be needed involving the stressing of 208 cable ends.

However, for design purposes the coefficients used were $K = 0.0001$ and $\mu = 0.15$. Actually then, only 840 tons of prestressing had to be installed necessitating only 144 cable ends to be stressed. Thus, the trial and projection method eventually saved 8 percent in materials and 30 percent in stressing operations.

Installation

Rigid helically corrugated sheath made from 24-gage corrugated steel sheet were installed, with diameters of 4 and 4½ in. Splicing was achieved by sleeving adjacent pieces with an overlapping section of slightly larger diameter.

The length of pieces varied from 5 to 20 ft, varying upon location of individual cables and the position of particular obstructions. The sheaths were placed from the main (middle) deck of the moving stage which also contained the main slipform and served as a platform for placing reinforcing steel and inserts, and depositing and vibrating the concrete. However, the working space was extremely constricted because the lifting equipment, as well as one upper deck, was located above this platform. Because of these obstructions, ducts could not be easily removed or similar measures taken. Hence, this was one reason for using relatively short lengths of sheath units.

Various alternate materials for the ducts and installation methods were considered. For example, corrugated plastic tubes, semi-rigid ducts, and the forming of a hole in the slipformed walls by means of a mandrel, were considered initially. However, rigid sheaths were selected because they were strong, were easily handled and installed, and were relatively cheap.

Despite rough handling, the ducts were not damaged and the blockages created by denting, punching or burning and other mistreatment were few.

Each cable duct was provided with a cap after every new unit was installed to prevent foreign material from entering and blocking the ducts. In retrospect, this measure proved quite successful except for four ducts which became clogged during construction. Three of these ducts were subsequently cleared by using a flexible shaft diamond drill. This instrument was very effective in removing concrete chunks (some 400 ft deep) from the ducts. Nevertheless, this drilling operation was very expensive and caused many delays. Therefore, not all the ducts were cleaned out in this manner.

Fig. 12 shows a series of capped sheaths while Fig. 13 depicts the installation of duct sheaths from the "moving deck."

The joints of the sheath were all wrapped with tape immediately after placing. This technique prevented cement paste from entering.

Grout vents were installed in pairs at

regular intervals of 100 ft, with a distance of 5 ft between the two vents of each pair. The lower vents of each pair would then serve as a control outlet for the grout pumped in from below, while the upper one formed the inlet for the subsequent grout lift above.

Fig. 14 shows the installation of grout vents.

A very important task was to uncover the grout vent openings immediately after the slipform operation before the concrete hardened over the pipe heads.

The installation of cables started immediately on completion of the respective walls. First the anchor heads and base plates were placed.

The strands were sleeved-in from the top, one at a time. First, they were unwound from the reels and then sleeved through a specially designed rig into the top anchor head. The speed of the

Fig. 12. Series of capped sheaths at base of Tower shaft.

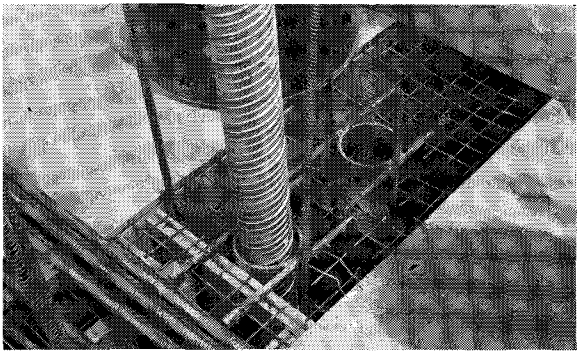

Fig. 13. Installation of tendon duct sheath from "moving deck."

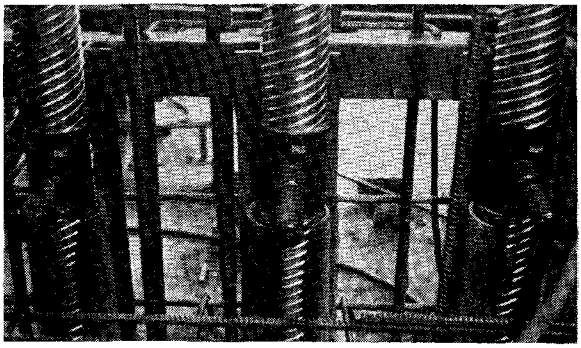

Fig. 14. Installation of grout vents. They were done in pairs at regular intervals.

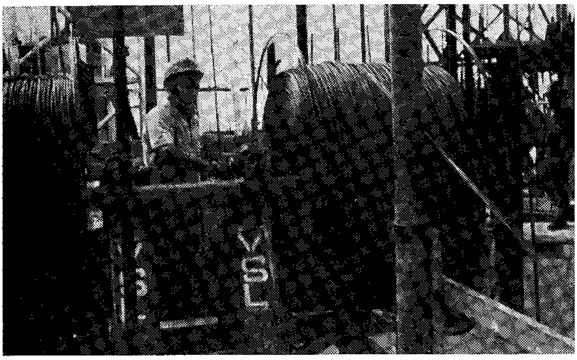

Fig. 15. Feeding strand directly from reel.

strand when descending into the hold was mainly governed by its own weight and controlled by a manual brake applied at the reel (see Fig. 15).

This operation required considerable skill on the part of the prestresser. Depending on the speed and other circumstances, the strands have a tendency to jam in the duct before reaching the lower end. These tie-ups had to be undone which at times caused considerable delays.

Although the sleeving-in of strands by this method was generally successful in that strands could be placed at a high rate, the process can be dangerous at times. For example, the strand had a tendency to coil up at the lower end of the duct when brake control at the upper end failed and the speed of the strand was mainly governed by its own weight.

Various devices and speeds of feeding were used for this operation. The most successful procedure turned out also to be the simplest. In this method the strands were led directly from the reel to the anchor head. Control was maintained by a man-operated brake at the revolving reel.

Some stressing difficulties were related to the placement method. For example, the consecutive insertion of strands at high speed can lead to coiling up of strands around each other inside the duct to form a knot in zones of sharp curvature.

In one cable group, the location of this knot was sufficiently close to the stress anchor head (at the base) to prevent stressing from the bottom end. The knot would have been pulled towards the stress anchor head, the entangled strands not being able to free themselves for their final position in the individually-sleeved anchor block. Strands then became kinked and snapped due to concentrated overstress.

The stressing operation then had to be moved to the top end for this group of tendons where it was successfully executed though with considerable delay. The lost strands were replaced in adjacent ducts (see Fig. 11).

Stressing procedure

The stressing method used depended on whether the tendon was stressed at the top or the bottom. For bottom-stressed cables, there was a region of sharp curvature just beyond the anchorage that caused rapid stress change in the tendon as it rounded the bend.

This had the effect of absorbing much of the anchorage "set effect" that is typical of individually wedged multistrand systems. There was, therefore, little to be gained by "overstressing" or overpulling the cable and then rebasing it.

Stressing was, consequently, very straight forward; the cable was pulled to a specified pressure at which point the wedges were easily set.

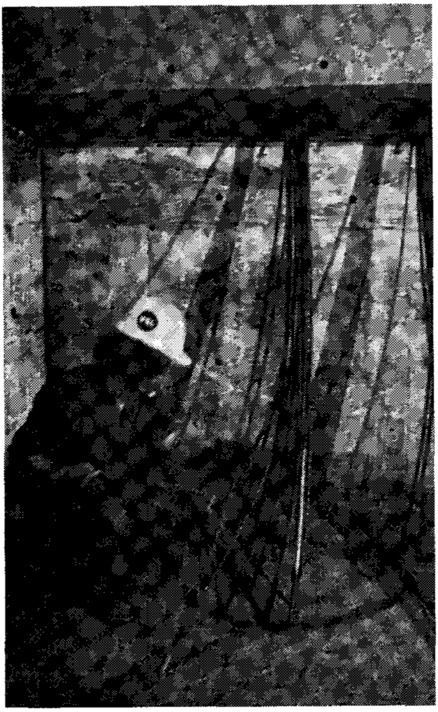

Fig. 16. Strands arrive at base through 1500-ft long duct.

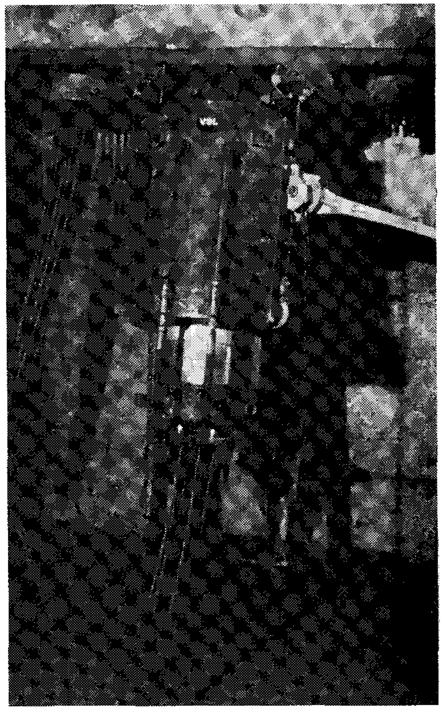

Fig. 17. Closeup of stressing in protective cavern.

Fig. 16 shows newly arrived strand at base of Tower while Fig. 17 shows the stressing operation in the protective cavern.

For the tendons that were stressed at the top end, the situation was different, since there was no adjacent zone of sharp curvature. Overpulling was advantageous here in getting the point of maximum stress to move a considerable distance down the cable.

Tendons were pulled to a specified pressure at which point the wedges were set. The complete anchor head was then pulled a specified distance and then released back through this distance.

Fig. 18 shows the stressing operation from the top anchorage.

The long stress elongations made it necessary to reset the jacks many times. For cables over 1000 ft long, the effec-

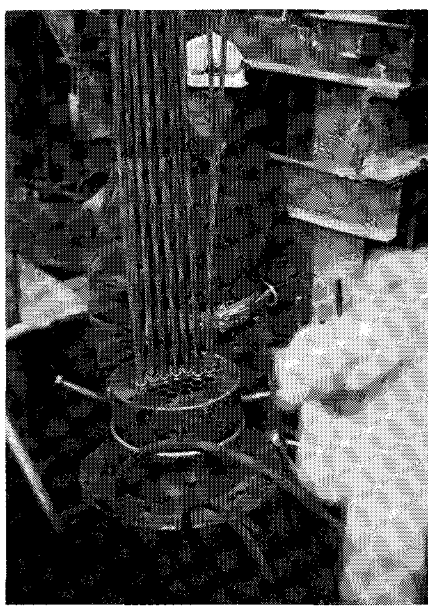

Fig. 18. Stressing tendons from top applying back-up procedure.

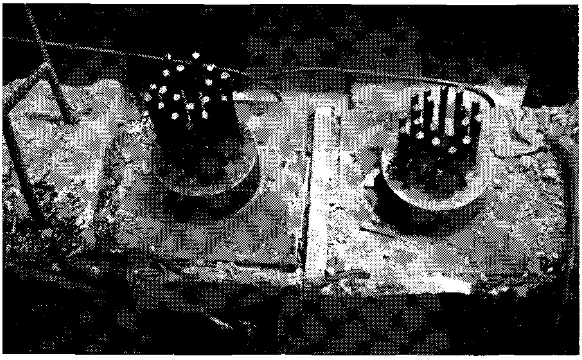

Fig. 19. Top anchorage at termination of wall.

tive stressing time was from 2 to 3 hrs when stressing could be executed from the base. However, when stressing took place from the top anchorage, the efficiency rate dropped sharply to 5 to 6 hrs for stressing one unit.

Fig. 19 shows the top anchorage at the termination of the wall.

Grouting

Design considerations—The grouting of long vertical ducts inside concrete walls is a relatively new procedure. It mainly evolved in connection with nuclear power stations and lately with offshore oil containers.

The use of grouted post-tensioning as opposed to ungrouted tendons provides many advantages which will not be discussed in this paper specifically. However, they are so important as to make grouting imperative for tall structures such as the CN Tower. Hence, the technical problems involved in vertical grouting had to be fully explored.

Questions were mainly related to three aspects:

1. The fluid grout would, prior to hardening, subject the concrete to substantial hydrostatic pressures, the effects of which could not be assessed with certainty analytically.

2. Construction of the Tower shaft would cover several months, extending into the cold season. During this severe weather grouting could not take place because of the extremely low temperature of the Tower walls. Hence, tendons that had to be placed and stressed as work proceeded, would be left unprotected by a grout enclosure. Consequently, a corrosion problem had to be anticipated and counteracted.

3. Literature on the subject suggested that large quantities of bleed water might be expected at the top of each grout lift. This question was therefore very important and had to be resolved with field tests.

The behavior of fresh grout under hydrostatic pressure was not well known during the design stage. Therefore, a series of tests were performed to explore its setting characteristics, its most suitable chemical and physical composition, and its tendency to produce bleed water. Basically, the composition was to be that of normal grout used for injecting post-tensioning cables, a mixture of Type 10 (normal) portland cement, water, a water-reducing agent, and an expansive agent.

The field tests were to include all known parameters in relation to actual construction conditions. In retrospect, this proved to be the correct approach because most of the uncertainties were resolved at a rather modest expense in terms of time and money. The total cost of the testing was of the order of 1 percent of the overall cost of the Tower shaft post-tensioning. This expense was justified in view of the serious consequences caused by unforseen difficulties.

Testing—The first experiment consisted of a series of tests using 24-gage rigid ducts (same duct material intended for use in the Tower). The ducts were 100 and 200 ft long and were mounted on a 200-ft guyed scaffolding tower.

Figs. 20 and 21 show the full-scale experimental setup for testing the grouting system.

At first, a water-cement ratio for the grout of about 0.44 was specified. Unfortunately, this ratio, when combined with the type of mixing equipment used (a colcrete colloidal mixer), produced a grout that was excessively viscous. Pumping pressures became excessively high and there was too little time between mixing and the time when the grout was no longer pumpable.

Hence, an increase of the water-cement ratio to about 0.50 became necessary. This mix resulted in a flow cone time of approximately 12 sec., allowing for efficient pumping. An average strength of just over 5000 psi was achieved. This strength was considered satisfactory since the main function of the grout was to be for corrosion protection.

The water-reducing agent (in liquid form) was found to perform satisfactorily.

Expansive agents were also added in some of the initial tests. The intent was to counteract shrinkage and separation from the steel sheath. However, due to the hydrostatic pressure of the 100-ft high grout lifts, a very undesirable phenomenon occurred. The gas produced by the expansive agent (in this case aluminium powder) was forced out of the mix and pressured to the top of the grout lift. There it collected in large-sized bubbles which, when hydration of the grout had occurred, resulted in a section of grout that was of an extremely porous and loose composition.

This portion extended for about 5 ft in height and, though its strength was

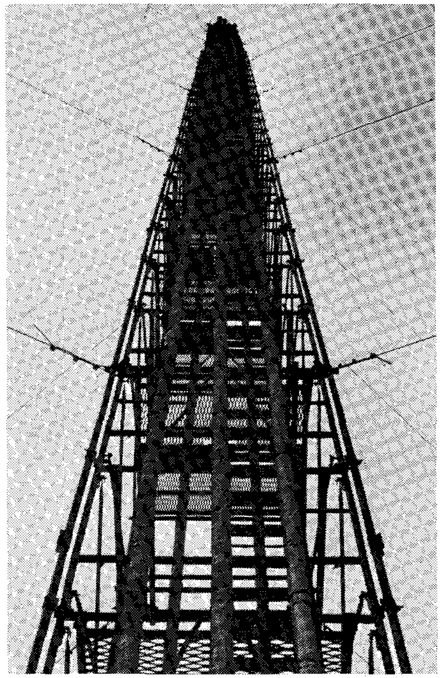

Fig. 20. Full-scale testing of grout.

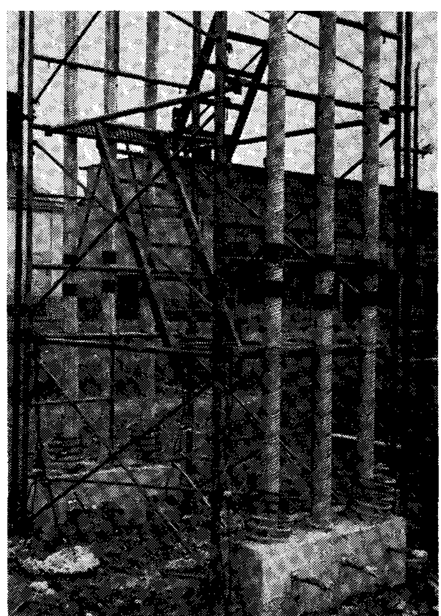

Fig. 21. Closeup of grout test in fills at base.

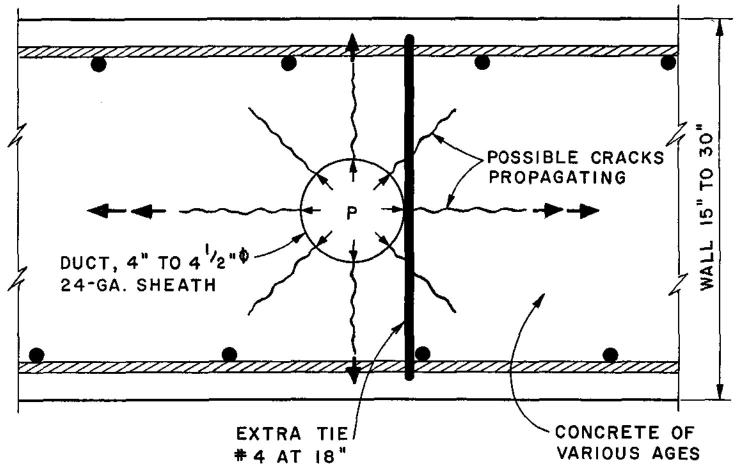

Fig. 22. Hydrostatic grout pressure.

sufficient to sustain the hydrostatic pressure of the subsequent grout lift above (about 100 psi), it was completely insufficient for structural purposes and corrosion protection. Consequently, the use of gas-producing expansive agents were abandoned and tests without the chemical additive proved satisfactory.

The tests also established that substantial amounts of bleed water accumulates at the top of a lift. This phenomenon, however, was much less pronounced in the actual grouting of the Tower, although the precise reasons for this are unknown.

Subsequent to the testing, various portions of the grouted ducts were opened and inspected. It was found that with the relatively viscous grout resulting from a water-cement ratio of 0.45, incomplete filling and wire coverage resulted. In some cases cavities in excess of 1 in. were present. This was judged to be unacceptable and so a water-cement ratio slightly more than 0.50 was used and proved satisfactory. The strength obtained from the two mixes was also measured, from 2-in. cubes (see Table 2). It was, as expected, considerably lower for the more fluid mix which was, however, considered a fair exchange for better corrosion.

A second series of tests was performed to investigate possible problems, especially the resistance of the concrete Tower walls against splitting. With fresh grout inside the ducts, considerable hydrostatic pressure would exist.

The behavior of a brittle body under hydrostatic pressure in an enclosed cylindrical surface is very complex. It is suffice to say that an analytical method which could predict crack propagation in time and space was judged most unreliable. Consequently, it was decided to conduct a series of tests to simulate

Table 2. Summary of average strengths of two mixes

Water-cement ratio	Flow test, sec.	7-day average strength, psi	Remarks
0.45	13	5500	Insufficient filling and wire coverage; pumping difficult
0.53	10	4700	Satisfactory

Note: The average 28-day strength for grout injected into the Tower cable ducts was over 5000 psi.

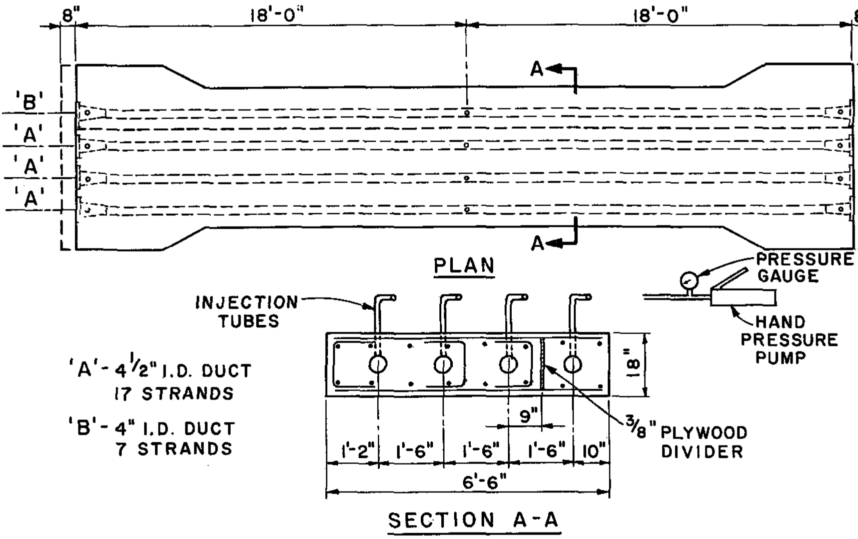

Fig. 23. Horizontal grout test panel.

the actual conditions in the Tower walls as closely as possible (see Fig. 22).

A test panel (see Fig. 23) was constructed and subjected to the longitudinal compressive stress that would exist in the Tower walls at the time of grouting (an average value of 1000 psi was chosen). Reinforcement was provided reflecting conditions in the critical (thinnest) portion of the wall that would be subject to grouting pressures.

The test panel was divided into two separate strips by building paper. One of the strips was transversely reinforced to verify the effectiveness of this in stopping the propagation of cracks while the other did not have any ties across the probable splitting region.

The inside of the ducts was pressurized with water and tested to destruction (splitting of the concrete). The splitting pressure varied from 300 to 500 psi. It was found that transverse reinforcement effectively stopped cracks from extending over longer distances.

Based on these results the following provisions were introduced into the design and construction procedure:

1. Specify a maximum height of grout lifts not exceeding 100 ft. (This height can be pumped with a maximum pressure at the pump of little over 100 psi.)
2. Provide a pressure relief valve at the inlet point which was to be set about 100 psi to prevent accidental overpressurization. (This relief valve was, however eliminated when grouting was executed after about 1 year when the concrete of the walls had reached its final strength.)
3. Furnish transverse reinforcing steel of one No. 4 bar at 18 in. spacing for every cable.

As a result of these procedures no splitting of the wall concrete was observed during construction.

Procedure and observations—The grouting operation was based on the principle of batching the grout near the inlet elevation for each lift. This entailed the use of a vertically movable platform to support the grout mixer, cements, water, and other materials.

The vertical grout test (described above) had shown that there was a possibility of substantial problems from excess bleed water. Accordingly, a double vent system with a 5-ft vertical separation between outlet and inlet vents was adopted. The idea was that up to 5 ft of bleed water could be drained back out after each lift. The total quantity of free water in the ducts would thus stay within reason.

In practice, however, the bleed water was relatively small and was not cumulative with respect to successive lifts, i.e., they did not vary markedly with the length of cable.

The top of each cable was normally provided with tall, large diameter, standpipes to collect the bleed water and to allow the accumulation of good grout under the bleed water.

The shortest cable group (180 ft long) showed bleed quantities equivalent to between 5 ft (in one case) and 14 ft of water in the ducts. The longest cable group showed similar quantities of bleed water. (Note that this observation appears to contradict experience reported in the literature by Morris Schupack.)

A variety of investigations were made during the grouting to confirm this peculiar occurrence. The conclusion was that the large amount of segregation expected was not, in fact, occurring. No proven explanation of this was actually found. However, the speculations are based on the following:

1. The extraordinarily dry condition of the ducts and strands prior to grouting was not foreseen or replicated in the initial tests. The dryness resulted from the application of a "dry air" corrosion prevention technique throughout the winter preceeding grouting.

2. The water transport mechanism assumption presumes that free water at the base of a grout column can infiltrate between the outer wires of the strand. It can then rise upwards in the space between the outer wires and the core wire driven by a hydrostatic head differential due to the difference in specific gravity between water and grout. It is assumed that the suspended cement particles are too large to get into this space.

To prevent this mechanism from taking place would require that the space between the outer wires is either reduced to the point that it clogs quickly with cement particles or increased to the point where it is fully infiltrated. Since the space between wires does, in fact, become smaller when strand is stressed and since previously reported tests seem to have been done with unstressed strand, the former explanation seems more likely. Furthermore, the constant quantity of bleed water irrespective of cable lengths would seem to indicate that, after a certain amount of water escapes, the space becomes clogged and no more water can be transported away.

It is, therefore, concluded that the water transport mechanism, that has been found by others and that was also confirmed in our own tests, probably has a limited capacity to remove water from a grout column. Since the mix remaining is comparable to that used in grouting most structures, it is felt that the lack of apparent bleed water has not been detrimental to the structure.

Corrosion protection

Grout was to be used as the main agent to protect the strands against corrosion. However, since most of the cables would have to be placed during the winter, cold temperatures would have precluded proper setting of the grout. This left the tendons unprotected and thus a means had to be found to protect the strands against corrosion.

The use of alcohol to fill the duct voids and various types of coatings were investigated but were found to be impractical or insufficient.

The method chosen was a dry-air system that eliminated moisture until the grouting could be done in the summer of 1974.

The specification required that the equipment supply 400 cu ft per min. of oil-free dry air at ambient temperature, but with a dew point temperature of − 100 F (− 55 C). Standard components were used, which included two compressors coupled to two heatless drying systems and filters. All this equipment was housed in a 40-ft insulated trailer adjacent to the job site.

The system operated as follows: the air is compressed and then passed through a filter to remove oil and other particles whereupon the air is again compressed and refiltered. It then passes through the final dryer where the remainder of the moisture is removed. Activated alumina is used as the drying agent.

A large-diameter polyethylene pipe connected the air-drying system to the base of the Tower, where smaller pipes connected the main pipe to the lowest grout vent in each duct. All intermediate grout vents were closed off, an operation taking 2 weeks, since all work had to be performed from swing stages.

At the top of each duct, a fiberglass or metal cover was placed over the exposed anchor head.

The extremely dry air was then pumped into the bottom end of each duct. A measured orifice at the top of each duct ensured up to three complete air changes per hour. When eventually the fiberglass covers were removed from the anchor heads and strand, the steel was found to be uncorroded. On this basis, it was concluded that the entire length of cable was likewise uncorroded because the top end is where most moisture would gather, thus inducing corrosion.

Auxiliary Stressing of Anchorage Zone

At the top end of the wing sections (coinciding with the base of the upper accommodation levels), three conditions arose:

1. The top anchorage of the largest group of post-tensioning units [3 x 10 cables at 700 kips (318 tons) nominal stressing force] was to be anchored at the exterior edge of the wing sections at a distance of about 10 ft from the central hexagonal part of the Tower. However, at the time of stressing, the central Tower wall would not carry substantial loads. Hence, a situation existed where high stresses would be introduced at the outside edges of a multicellular truncated tower (see Fig. 24). This constituted a large-scale anchorage zone with high local tensile stresses.

2. Later, during construction the stress condition would reverse itself. Higher loads would be concentrated in the central portion due to the weight of the upper part of the Tower walls in addition to the stage post-tensioning (see Fig. 25).

3. The erection and concreting of the supporting bracket walls for the upper accommodation levels resulted in substantial horizontal loads on the same portion of the Tower shaft (see Fig. 26), before the supporting ring beam would be post-tensioned and act as the final lateral support of the cantilever walls. This, in effect, again reversed the stress condition.

All three conditions produced high local radial tensile stresses. The transfer of these stresses to the central hexagonal cell had to occur within the bounds of the slipformed Tower walls. Hence, the need for prestressing was indicated.

A system of stress bars was selected that allowed the placing and stressing of the tendons in stages. In the first stage, the stress bars were installed within the slipformed walls to counteract the tensile stresses due to anchorage of the Tower post-tensioning cables. In the second stage, coupling was added as part of the scaffolding for the upper

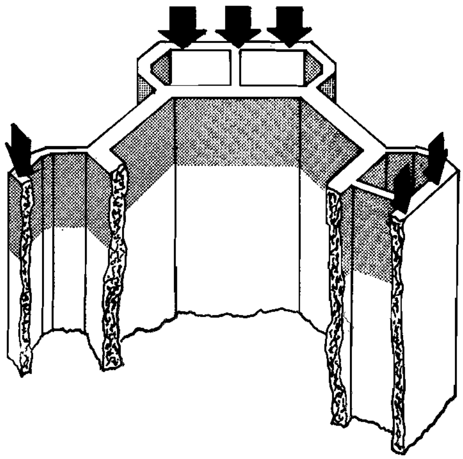

Fig. 24. Stage 1 loads concentrated at outside edges of truncated Tower. Note that shaded areas denote tensile stress.

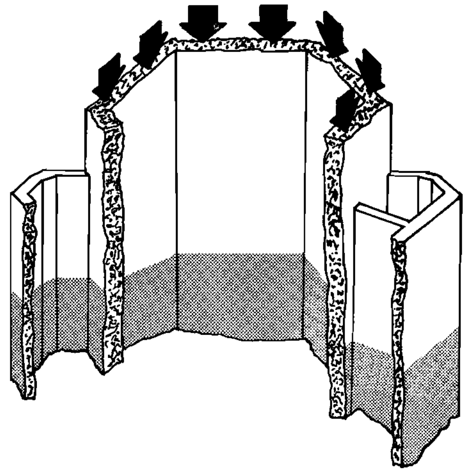

Fig. 25. Stage 2 loads concentrated in central portion of truncated Tower. Note that shaded areas denote tensile stress.

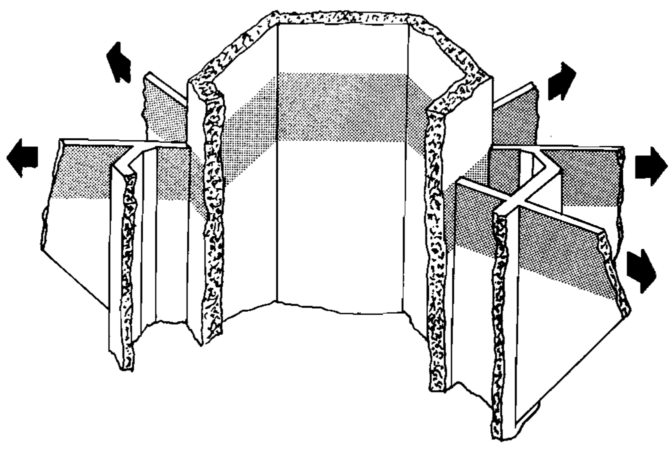

Fig. 26. Stage 3 horizontal loads from bracket walls. Note that shaded areas denote tensile stress.

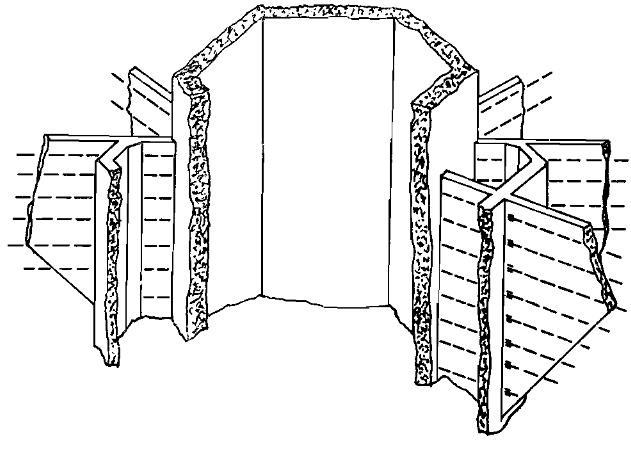

Fig. 27. Arrangement of stress bars for lateral support.

accommodation. This provided the required temporary lateral support for the weight of the concrete walls (see Fig. 27).

Stressing was carried out for each stage as soon as the concrete reached a sufficient strength. Since completion, no cracking in the concrete has been observed.

Ring Beam (Upper Accommodation)

The main lateral containment of the upper accommodation structure is formed by a post-tensioned concrete ring beam at the top of the bracket walls (see Fig. 28).

The ring beam acts within the floor of the second level (outdoor observation) and was constructed as the last element of the concrete base structure for the "pod," in the 1974-1975 winter.

The main function of the ring beam is to resist direct tension with only minor bending due to the weight of the floor it supports. Consequently, the post-tensioning was arranged in a concentric and symmetrical manner.

The ring beam forms a twelve-sided polygon, reflecting the radial loads it will withstand. Its final post-tensioning force, after all losses, was estimated to be 2400 kips (1090 tons) and the ultimate tensile strength of the cables was specified as 4200 kips (1900 tons). This force is arranged in twelve cable units, consisting of 49 wires each, with an ultimate tensile strength of 241 ksi (170 kg/mm^2), so that in every section of the beam six units are present.

The most delicate design problem was to arrange the tendon units in a symmetrical pattern all around, including all stress transfers due to friction at stressing. All anchors had to be located towards the inside of the beam. This re-

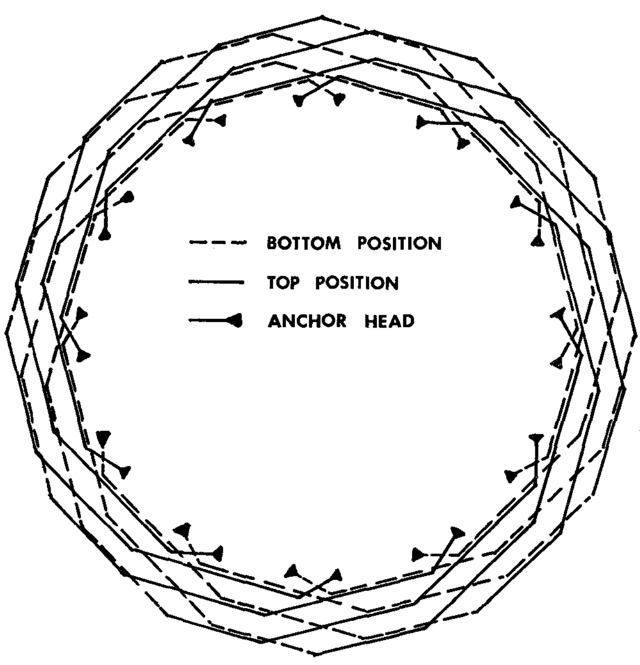

Fig. 28. Layout of tendons for ring beam.

sulted in a rather intricate placing pattern which is shown schematically in Fig. 28.

Each cable is placed in identical position, reaching around 180 deg, and staggered every 30 deg. All 24 anchors were stressed sequentially to ensure the closest approximation of symmetry at all times. Stressing elongations were found to be very uniform, with a maximum variation of about 5 percent on an average elongation of 10½ in.

Closing Remarks

The CN Tower, with the exception of some portions designed in structural steel, is a fully-prestressed concrete structure. Essentially, all prestressing (or post-tensioning) is arranged concentrically and designed to disallow tensile stresses.

Thus, full prestressing has been provided within working load levels (as defined by estimates of dead and live loads and statistical 50-year wind loads). For loadings in excess of these levels, the structure will become partially prestressed and tensile stresses will occur in certain portions of the concrete.

All prestressing is essentially uniaxial. Therefore, the above statements must be qualified since tensile stresses will also occur at working load, transverse to the main stress direction. Most of these transverse stresses are, however, quite small.

For example, the shear stresses due to wind are greatly reduced because of the tapering shape of the Tower. The shape approaches the idealized shell structure shown in Fig. 29, which for a lateral load resultant coinciding with the apex of the tangential cone, does not produce shear stresses.

Equilibrium to the exterior lateral load is then held by the horizontal components of the wall stresses which are direct (normal) stresses but have an inclination equal to that of the wall.

Other sources of tensile stresses exist, and reinforcing steel was provided throughout for the effects of local bending, shrinkage, temperature, torsion, and other stress conditions.

Fig. 30 shows the CN Tower in May of 1975.

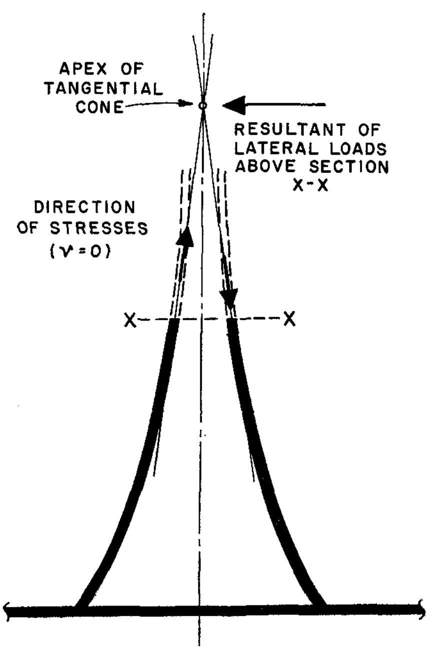

Fig. 29. Idealized Tower stresses.

Credits

Owners: CN Tower Ltd., Toronto, Ontario.

Architect: John Andrews International, Roger du Toit; the Webb Zerafa Menkes Housden Partnership, Toronto, Ontario.

Structural Consultants: Nicolet, Carrier, Dressel and Associates, Limited, Montreal, Quebec.

Manager Contractor: Foundation Co. of Canada, Toronto, Ontario.

Foundation Post-Tensioning: Conenco Canada Ltd., Toronto, Ontario..

Tower Shaft Post-Tensioning: VSL Canada Ltd., Stoney Creek, Ontario.

Auxiliary Stressing of Anchorage Zone:

Supply: Dywidag Canada Ltd., Toronto, Ontario.

Construction: VSL Canada Ltd., Stoney Creek, Ontario.

Ring Beam, Upper Accommodation: BBR Canada Ltd., Toronto, Ontario.

Acknowledgment

The authors wish to express their sincere gratitude to all participants in the CN Tower project for their help and collaboration in preparing this report. Particular thanks must go to Frank Tam, resident structural engineer, for documenting the pertinent records during the construction period.

A project of this magnitude can only be built when a very congenial and understanding atmosphere exists between all participants. This inspiring collaboration will long be remembered by the authors. Lastly, one important ingredient for this smooth teamwork was the owner's generous cooperation, without which monumental structures of this type cannot be successfully built.

A heartfelt thank you to all!

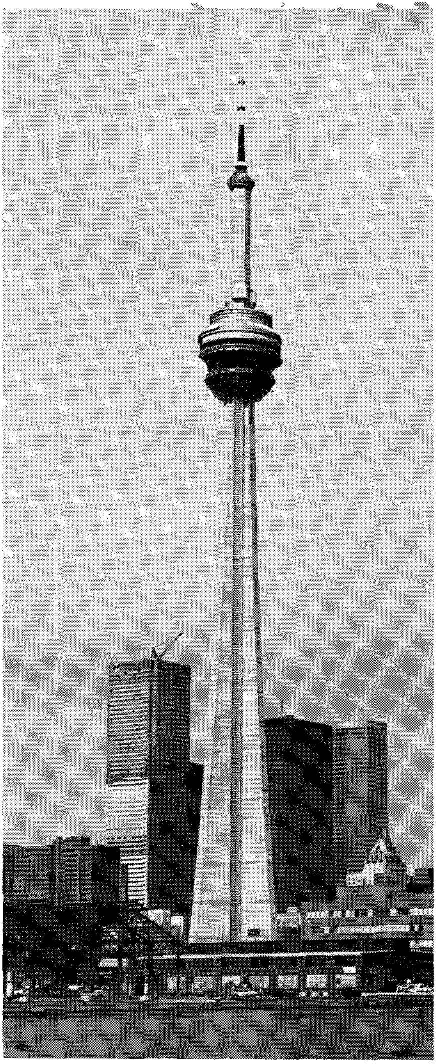

Fig. 30. CN Tower from waterfront (May 1975).

Discussion of this paper is invited. Please forward your comments to PCI Headquarters by October 1, 1976.