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Slender Concrete Structures — The New Edge

by Jacob S. Grossman

The special problems associated with tall concrete structures are reviewed and shown to be magnified in tall and slender buildings. The need for open panoramic views to accommodate the occupants of these slender structures penalizes the construction economy and tasks the engineer to provide adequate serviceability, while considering the perception of motion to be a prime design parameter. Possible solutions to these problems are introduced. Case studies of three slender structures with 10-to-1 aspect ratios are reviewed.

Keywords: bracing; cavity walls; cladding; computer programs; cracking (fracturing); creep properties; damping; dynamic loads; environments; flat concrete plates; gusts; **high-rise buildings**; lateral pressure; motion; reinforced concrete; rotation; **serviceability**; shearwalls; shrinkage; **slenderness ratio**; stability; stiffness; **structural design**; tube-in-tube; weight (mass); wind pressure; wind tunnels.

EVOLUTION OF DESIGN METHODOLOGIES
One consulting office's experience

In the early 1980s, developers discovered a quirk in the zoning laws that led to the production of a string of slender "sliver" structures in Manhattan. Morgan Court[1] is one such structure; built in 1983 at 211 Madison Ave., it is 330 ft. tall and only 32 ft. wide (see Fig. 1). This extreme slenderness ratio (over 10 to 1) came on the heels of an earlier breakthrough in the slenderness ratio. An office building at 780 Third Ave., with an 8.1:1 aspect ratio, dictated the development of the first diagonally braced tube concrete structure.[2,3]

Prior to designing these structures, a review of about 700 structures, a majority of them high-rise concrete structures (the product of 35 years' practice by the consulting firm of Robert Rosenwasser Associates P.C.), showed that the perception of motion had not been a problem with which to contend. The firm's practice utilized mainly flat plates or slabs, with the aid of some or no shearwalls and other frame elements (such as beams) to resist the lateral loads. Several parameters dictated the quantity of shearwalls provided in a structure: the various codes and design methodologies predominant at the time the structure was designed; the architectural demand for open spaces, free of shearwalls; the type of occupancy involved; and the economy of construction. Therefore, structures that accommodated placement of shearwalls received them (if safety first and economy second dictated their need). Similarly, structures which required false ceilings to hide mechanical apparatus (such as office buildings) would also accommodate beam drops (again, if safety and economy dictated a need). Otherwise, slab-frame action was utilized as much as possible because minimizing forming cost and maximizing freedom for architectural and mechanical requirements enhanced economy and improved occupant space.

In recent years, with the growing awareness that the East Coast is susceptible to moderate seismic action (though at a much lower frequency of occurrence than the West Coast), a mixture of semiductile frames with shearwalls have been used to enhance redundancy. Integrity reinforcing to minimize the possibility of progressive collapse, and the details for ductility[4] had already been incorporated for a quarter of a century. This design process, which does not increase construction costs measurably, will comply with future code requirements soon to influence construction on the East Coast.

WORKING WITHIN THE INNER CORE OF OUR "SPHERE OF KNOWLEDGE"

In pre-World War II structures, massive brick and stone cladding and masonry partitions undertook to provide redundancy in buildings. Often they became the second line of defense, aiding limber-frame structures in supporting both lateral loads, and most of the gravity loads. With the postwar introduction of cavity-wall construction, and shortly thereafter nonstructural sheet rock partitions, this second line of defense was removed. The problems that developed in the cladding of many structures were mainly attributed to the lack of separation between the cladding (cavity walls) and the

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structure's elements. These problems forced the engineering profession to expand its "sphere of knowledge" and to advance its construction and design methods so that the structures that were designed nested comfortably at the inner core of this sphere.

The "sphere of knowledge" of a topic — in this case the design and construction of buildings — is that volume of information about the topic which is known and which is fully understood. If through the passage of time, designs justify the assumptions made, we are situated within an "inner core" where a deisgn can proceed on similar structures without the need to "dot all the i's and cross all the t's."

In the early 1950s, the seemingly innocent substitution of nonstructural cladding for structural cladding caused the structrual profession inadvertently to step outside the bounds of its knowledge. The slow process of re-evaluation started shortly after problems were first observed.

The need for soft joints to separate structural and nonstructural members was eventually introduced into the New York City Building Code in about 1963. A horizontal soft joint every 40 ft became a code requirement. With the passage of time this was shown to be inadequate. Structures with this minimal provision have been and continue to be repaired throughout the city.

Another example of the industry's inadvertently stepping outside its sphere of knowledge was the introduction of deicing salts. This resulted in tremendous infrastructure renovations taking place in parking garages, bridges, and highways. Another example was the use of asbestos to fireproof structures — and the list goes on. The question of whether design assumptions fall within the inner core of our knowledge of construction can only be determined over time.

The structural profession finally realized that structures do move, that concrete columns do creep and shrink, that the material used for cladding expands because of temperature, moisture absorption, etc., and that it is best to separate the structural from nonstructural elements at every level. It took three decades to fully recognize the proper details for cladding separation and the need to separate the mechanical systems. It is still necessary to inform other trades of this need and to show them how to allow for vertical, as well as horizontal, movements (see Reference 4 for a thorough discussion of this subject). Such problems are present in all structures, large or small, and are greatly magnified in tall, slender structures.

SPECIAL CONSIDERATIONS FOR THE DESIGN OF SLENDER STRUCTURES

The long-term creep and shrinkage of concrete structures have been considered nemeses for many years. What was not immediately appreciated was that despite these faults, concrete also has some major advantages. These advantages are not immediately obvious because they have to do with the material's heavy weight and its susceptibility to cracking. One must actually *design* tall, slender structures to recognize the advantages this pair can provide.

In concrete structures designed using archaic codes (such as the pre-1970 New York City Code, which did not require wind forces on the lower 100 ft of a structure), as well as in structures designed for hurricane forces in Florida, complaints relating to the perception of motion were not received. Never having confronted the problem of limiting the perception of motion, there was no need either to deal with or become familiar with controlling the perception of motion in concrete structures. This was the scenario until about 1982, when the plans for a "sliver" concrete structure [with a major leap in aspect ratio of height to least depth—see Fig. 1(a)] was introduced. A potential for the perception of motion to become a problem was immediately recognized; previously, a five or six aspect ratio was considered extreme.

The question to be resolved was, "why are concrete structures (unlike steel structures) generally immune to the problems associated with perception of motion?" To answer this question attention must focus on three things: (1) the benefit obtained by the heavier material, concrete; (2) the formation of micro and larger cracks (which increase the damping ratio of a structure); and (3) the ability of a well-detailed concrete structure to have some of its members perform inelastically to further improve its damping ratio. Obviously, the larger mass and damping inhibit problems due to excessive vibration from developing and being perceived by the occupants.

Because of its over 10-to-1 slenderness ratio, 211 Madison Ave. was designed with due consideration given to the perception of motion. This structure is somewhat shielded and has large mass and increased damping, due to masonry cladding on the longitudinal north and south building faces situated on the middle of the block lot-lines. Even though only 330 ft tall, an aeroelastic model study was performed to verify its functionality. A thorough review was needed to allocate stiffness and sufficient mass to achieve, with a minimum structural penalty, the industry's standard of about 15 milliG for a 10-yr return period for apartment structures. The emphasis in this design was somewhat different than for other structures. Serviceability, perception of motion, fundamental periods, and damping were the values looked at more closely, rather than the more mundane set of parameters (such as stress, strength, and drift) that dictate the design requirements for structures with more common aspect ratios.

211 Madison Ave. is braced [see Fig. 1(b)] using a

centrally located coupled shearwall (allowing for elevator lobby penetration), which is the main structural member in the short direction. Shallow spandrel beams and flat-plate action of a few interior columns provide additional lateral-load resistance. The central shearwall utilized coupling beams to tie the southerly flange of the wall (framing the stair) to the center elevator shaft and the column flange at the north side of the structure. The stair concrete flange was eliminated at some upper levels [see Fig. 1(a)] to minimize the influence of the shearwall base rotation and to allow the introduction of additional frame members which, together with other frame elements, helped introduce a larger moment reversal into the top of the shearwall. The structure was made stable using its own weight at the lower levels by extending the north and south flange of the shearwall to engage additional exterior columns. 211 Madison Ave, has been in service for about 6 years, has experienced moderatly high winds from Hurricane Gloria in 1985, and has behaved very well to date.

In 1983, New York City zoning laws clamped down on "sliver" structures less than 45 ft wide, but left the door open for structures with wider bases. Since 1983, Robert Rosenwasser Associates P.C. has had the opportunity to design over a dozen tall structures with slenderness ratios over 8:1 (case studies for two will be presented later). With one exception, all of these slender structures disallowed extensive structural support along the structure's circumference. The obvious need for large exterior windows to help sell Manhattan views dictated smaller columns and larger spacing on the outside periphery of each structure. The main structural elements had to be hidden within each structure. While unobstructed views can be accomplished with only a moderate penalty in structures having aspect ratios less than 5 or 6, comparable unobstructed views are the source of major penalties in very slender structures.

A hint as to what causes these penalties can be obtained through experimentation. Abundantly available computer programs can easily point out the detrimental effect slenderness has on the chord action of frames and the deterioration of the stiffness of tall and slender shearwalls. Simple mathematics will also indicate that for tube-type structures, where most of the resisting elements are on the outside, a 5-to-1 aspect ratio will require about one-fourth the material a 10-to-1 structure of the same height and supporting the same lateral load will require to limit the drift to comparable levels. It becomes obvious that placing the material on the periphery is the proper way to develop more efficient tall, slender structures, but in most cases the architectural demands are contrary.

PERCEPTION OF MOTION

The frequency of windstorms and the acceleration caused by the wind are prime parameters associated with the perception of motion of tall, slender buildings. While acceleration a is the quantity that is easier to measure (and thus is used more frequently), to assess the perception of motion in a structure, it is possible that the rate of change of acceleration, "jerk," is the "motion" that is actually perceived. The windstorm-induced motion is assessed as the root-mean-square value corresponding to the peak 10 to 20 min of the worst windstorm expected to occur an average of once in 6 to 10 years. The peak accelerations are then estimated from the root-mean-square accelerations, using a peak factor.

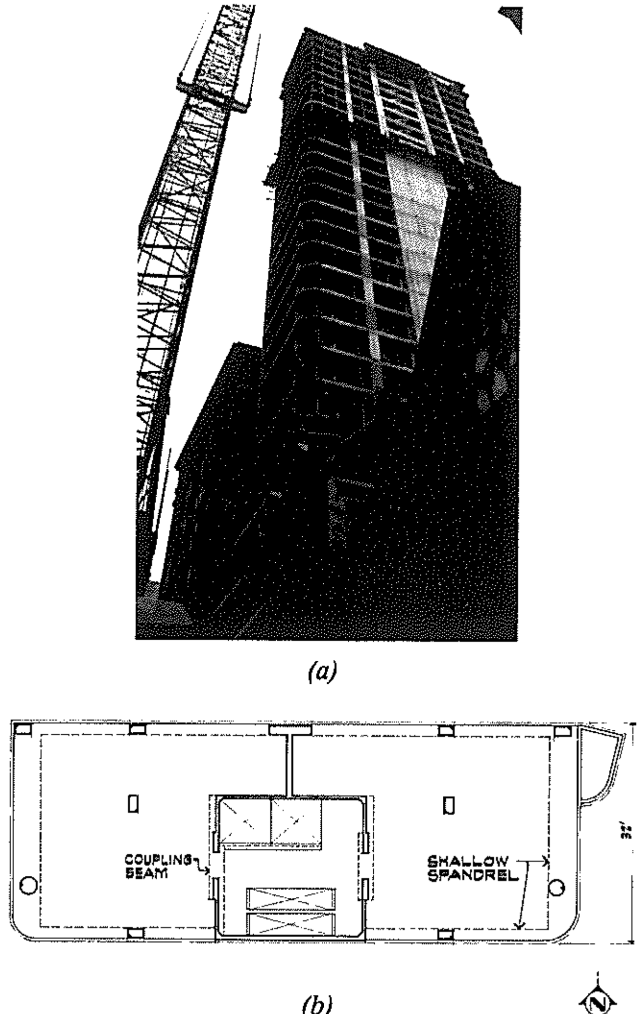

Fig. 1—211 Madison Ave. (recipient of the Concrete Industry Board's 1984 Residential Award): (a) south elevation; (b) typical floor plan (Photo: G. Gwertsman)

Experiments to determine human perception of motion thresholds were based on the natural period T of the structure versus magnitude of acceleration a, and/or the drift Δ caused by the wind forces. A host of other parameters (noise, visual effects, etc.) will also influence these thresholds (see Reference 5 for a thorough discussion of this subject). Interestingly, the thresholds of annoying accelerations (targeted at a minimum adverse-comment level of 2 percent of the involved occupants) has been established with the aid of developers who, in answering questionaires, indicated how large a percentage of occupants objecting to such "motion" can be economically tolerated (Reference 6).

For tall and slender structures, limiting drift (so that nonstructural elements are not damaged) may not provide the assurance that tolerable levels of motion will automatically be attained. Elementary physics indicates acceleration a to be proportional to the distance Δ to be traveled, and inversely proportional to the second power of the time, the period T it takes to travel this distance

$$a = \Delta\,(2\pi/T)^2 \qquad (1)$$

Here a is the maximum acceleration and Δ is the dynamic amplitude. It may appear prudent, as a first measure, to stiffen the building so that the sway will be reduced. Reducing sway by adding stiffness does two things that oppose and tend to cancel each other out when the reason to reduce the sway is minimizing the perception of motion. Though the added stiffness reduces the sway, it will also shorten the period of time T it takes the structure to complete a cycle of motion. Eq. (2) indicates that the period T is proportional to the square root of the mass M and inversely proportional to the square root of stiffness K

$$T = 2\pi\sqrt{M/K} \qquad (2)$$

Assuming that the structure behaves elastically, the deflection Δ is inversely proportional to the stiffness. Substituting $1/K$ for Δ, and $2\pi\sqrt{M/K}$ for T, in Eq. (1) indicates that the acceleration is not affected by the increase in stiffness but is inversely proportional to the mass. This relationship is also certified from $F = Ma$, which implies that for a constant dynamic force F, the product of acceleration and mass remain constant. Therefore, it seems that increasing the mass of the structure will reduce perception of motion effectively, while increasing the stiffness will not be effective. In reality, other complicating parameters are also involved and it is generally prudent (to reduce perception of motion) to stiffen the structure somewhat and increase its mass while doing so. In this case, the increase in mass will negate the reduction in period due to the added stiffness and allow the structure to sway with reduced accelerations. It has also been established[5] that somewhat larger accelerations can be tolerated when the fundamental periods are longer.

Another important parameter influencing perception of motion is the inherent damping of the structure. Concrete structures do possess larger damping capabilities to help stop the building motion quickly.

To review the control of perception of motion, an in-house developed computer program called TOWER (for three-dimensional structures) incorporated the procedures described in the supplement to the National Building Code of Canada.[7] These procedures indicate that increasing mass and/or stiffness has a positive influence on reducing the perception of motion. It also has been determined that it is sometimes economically impossible to provide sufficient stiffness and/or add enough mass to tall, slender structures to reduce the perception of motion to the established tolerable levels.

ADVANTAGES AND DISADVANTAGES OF INCREASING THE STRUCTURE'S MASS

Increasing the mass of a structure decreases the perception of motion because a larger period is maintained for the stiffer structure with the reduced sway. Unfortunately, every silver lining has a cloud. To cloud the issue, there are other hidden pitfalls lurking which deal with the performance of structures susceptible to lateral forces.

A measure of the lateral wind forces on a structure is a function of the mean average hourly wind speed and the fluctuating pressure caused by the wind's gusts. The interaction of the wind with the resisting structure is increased with a larger period, which will allow more wind forces to interact with the structure. Therefore, larger periods, a function of the larger mass, expose the structure to a larger accumulation of forces. The inertial forces developed as the structure moves are a function of the resonance response to the gusts loading and are numerically equal to the product of the structure's mass and acceleration.

Sometimes strong periodic loading can occur due to wake effects (such as vortex shedding) or instabilities (such as galloping and flutter). If the natural frequency of the structure becomes nearly the same as the vortex-shedding frequency, large-amplitude motion may occur, which can create occupant discomfort and even structural distress. However, the amplitude tends to be self-limiting. If an instability like galloping or flutter occurs, structural damage and collapse may follow. In such cases it is necessary to change the structure's period to insure that the periodic loading does not match the structure's period and that the instability does not occur for winds expected during the life of the structure.

The onset of instabilities and the severity of resonance are affected by the mass and damping. A larger mass at the top of a structure (producing a larger modal mass for the building in its fundamental modes of vibration) could help reduce harmonic dynamic interaction between the wind and the structure by reducing the amplitude of motion. This harmonic interaction could, for example, cause vortex shedding to lock in with the structure's across-wind period, which would considerably increase the total lateral forces and the discomfort level. Reducing the amplitude of motion by additional mass or damping reduces the chances that lock-in will occur for structures where the vortex-shedding frequency matches the building's natural frequency at wind speeds lower than the design wind speed. For example, lock-in is a more serious concern for steel structures than concrete structures.

WIND-INDUCED FORCES

Winds vary in strength, structure, and frequency for different directions depending on climatological factors, the upwind terrain, and adjacent structures. Buildings also respond differently to winds from various directions due to the shape and orientation of the building and its relative stiffness in various directions.

Thus the shape, stiffness, and orientation of the building (with regard to strong, frequent winds) affect the overall performance of the building with regard to the perception of motion and how frequently that perception occurs. While a building does sway slowly in a quasi-steady manner in response to high-period variations in wind speed, most of the dynamic responses of a building occur at its natural periods and are manifested as a resonance response to gusts or wake effects (flow separation and reattachment; vortex shedding), with periodicity near the natural periods of the building. How the energy available in the forcing function is distributed across a range of frequencies is important for determining this major component of the building's response.

Much of the wind's energy (arising from gusts or wake effects) is concentrated at relatively high periods. Stiffening the building (thus reducing its natural periods) will reduce the amount of force the building attracts through resonance because the periodic forcing function (with period near the natural period of the building) is reduced. Conversely, adding mass without increasing stiffness will produce a building with higher natural periods, which will attract more force and hence the beneficial effect of the increased mass is somewhat reduced. A larger mass will also increase demands due to P-Δ effects.

To summarize, the total lateral load caused by wind action on the structure can be visualized as the sum of three separate contributors, one static and two dynamic in nature. The first contributor is the "mean" static wind load, which can be measured quite accurately by wind-tunnel testing of a rigid model. The second contributor is caused by the fluctuating part of the instantaneous wind pressure. In large structures, there is a lack of correlation of the wind pressure over the full height of the exterior surfaces so that this dynamic action (unless it is in resonance with the structure) is generally small in comparison to the third contributor. This latter is a function of several minutes of wind action, rather than the precise wind distribution of any particular instant, and is caused by the inertial forces developed as the structure sways. This inertial force is equal to the product of the mass and the acceleration of the structure. In very tall and very slender structures, the dynamic wind loading (the sum of the second and third contributors) is often much larger than the mean static loading. To assess these contributors at the present time requires testing a model of the structure in a wind-tunnel laboratory.

DEVELOPING DESIGN AND DYNAMIC RESPONSE CRITERIA USING A WIND TUNNEL LABORATORY

To assess the dynamic response of a structure using a model of the structure in a wind tunnel, it is necessary for the design team to provide the laboratory with the following information: the location and shape of the structure; the mass distribution in the structure; the fundamental periods (torsional and in the two major orthogonal axes); and the deformation curves and the damping anticipated.

Needless to say, it is necessary to first design the structure based on available information and local code requirements. For concrete structures, it is also necessary to estimate the stiffness deterioration as a function of the wind load levels. Because these wind loads are not yet known, code-prescribed forces must be used initially.

An aeroelastic model of the structure can then be constructed (using the initial design results) and tested. This model must simulate the flexibility, damping, and mass of the prototype structure to provide detailed information on the movement of the structure as well as the dynamic loads generated by this motion. The construction and testing of such a model is time-consuming and expensive. Should initial results indicate revisions to the structure, the model must be revised and tested again.

Recently, a more efficient and economical means to estimate wind loads has been used in wind-tunnel laboratories. The wind mean and dynamic forces on the structure are measured, using a lightweight rigid model of the building mounted on a high-frequency force balance device. The dynamic inertial response of the structure is then analyzed with the aid of computers. Next, the measured dynamic wind and computed inertial forces are combined to indicate the peak dynamic force. The peak dynamic force added to the mean force will provide the required design load.

This simplified method is suitable when torsional effects are small and when the building motion itself does not affect the aerodynamic forces.[8,9] This system is attractive to designers because of the advantages of early reporting and mathematical projection of results to several alternative structural systems (each with its own mass stiffness and damping), without additional laboratory testing.

ESTIMATING DYNAMIC PROPERTIES

The estimation of the fundamental periods of a concrete structure (needed to estimate wind forces and accelerations) is a study of probability and cannot be predetermined with any great certainty during the design process. The reasons for this are numerous. First, the theory may not match reality. Second, large variations can be realized from design assumptions for the modulus of elasticity and the section modulus of the inplace concrete member due to construction procedures, stripping operations, creep and shrinkage, etc., which will affect initial stiffness and, thus, the initial periods. Third, with additional cracks, stiffness is reduced (but on the plus side, the damping of the structure is increased). Fourth, disengagement of the partitions will reduce stiffness and damping. Concrete gain in strength, with time, will somewhat diminish the reduction in stiffness due to other causes, etc. Therefore, it is difficult during the design stage to predict the dy-

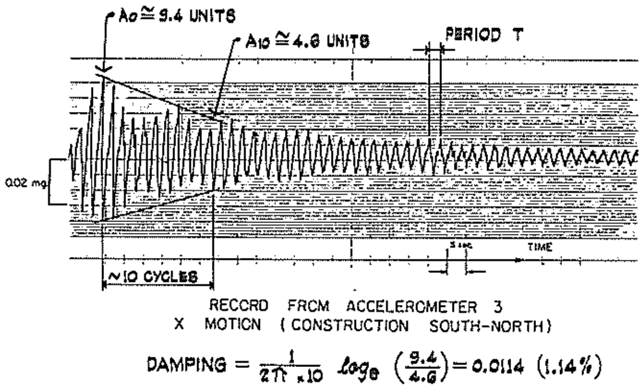

Fig. 2(a)—Field measurements of ambient periods and damping (courtesy RWDI, Guelph, Ontario)

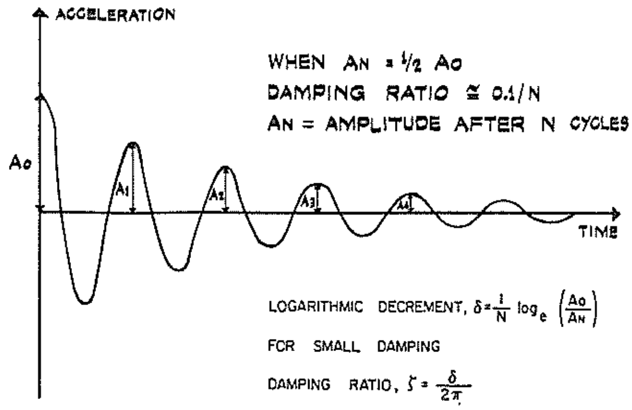

Fig. 2(b)—Logarithmic decrement method (courtesy RWDI, Guelph, Ontario)

namic properties of a structure to feed this information to the wind-tunnel laboratory, which then estimates the design requirements. However, this must be done, based on the best available predictions for the structure. It is easier and more accurate to obtain this information during the construction period, or shortly thereafter. (The reason this information might be needed after the structure is built will be reviewed later.)

To determine natural periods and damping, field measurements can be made using accelerometers and recording instruments (utilizing either the crane to vibrate the structure, or leaving these instruments in place to await some wind action). Small ambient motions are measured and the dynamic properties obtained are then projected for larger motions that account for stiffness deterioration.

Fig. 2(a) represents field measurements derived using a construction crane to move a slender structure nearing completion. Ambient periods were estimated by measuring the time lapse between ridges. The damping can be estimated using the logarithmic decrement method for small damping [Fig. 2(b)]. The number of cycles N required to diminish the amplitude to half-size are counted. The damping ratio can then be determined.

The extrapolation of larger natural periods and damping, which accompany larger wind loads, is again a study of probability that depends largely on the history of events the structure has already encountered. The field measurements of Fig. 2(a) were taken when the structure was partially completed (at its fifty-fourth floor). Similar measurements were taken a few weeks later when the structure reached its sixty-sixth floor and after Hurricane Gloria exerted intermediate wind forces on the structure (approximately two-thirds clad at that point).

The second set of ambient field measurements indicated a slight increase in damping (to 1.25 percent), but did not indicate larger periods than those anticipated by projecting the initial set of ambient period measurements of the incomplete frame to the almost completed structure. It is assumed that the crane could not provide large enough forces to detect stiffness degradation that may have occurred during the hurricane.

Limited information is available on concrete structures' reactions to various levels of wind force. A gleam of what may happen when design forces are in action can be deduced from the U.S. Department of Commerce report on the 1971 San Fernando earthquake, as well as from some small-scale reinforced concrete frame studies conducted at the University of Illinois at Urbana-Champaign.[10] The U.S. Department of Commerce report, which measured actual structures indicated that ambient (pre-earthquake) natural periods were lengthened considerably during the seismic event. Ambient periods after the seismic event showed a partial recovery but remained larger than the preseismic values. The University of Illinois study indicated damping ratios greatly dependent on the largest event previously encountered (damping ratios recorded for the largest event were also observed for subsequent smaller lateral loadings). The natural periods exhibited similar recovery trends to those observed in the San Fernando earthquake. From these observations it can be deduced that the fundamental properties of the structure will be altered permanently if an inelastic excursion (due to large wind action) should occur. An elastic response to smaller wind forces might alter the properties only minutely.

It is necessary to predict both the damping and the periods for a range of lateral loads that the structure may encounter throughout its service life during the design stage. As stated, both the damping and the periods are a function of the magnitude of the lateral loads. The engineer must predict a range of periods and damping ratios that includes the most probable low and high values, as well as the range for the more common 30 to 50 mph wind forces for which perception of motion is being reviewed. The observations of the preceding paragraph indicate that, for actual structures (which are designed for 100-year return forces), ambient measurements of both damping and periods should be only slightly increased when evaluating the perception of motion. At design load levels it is logical to assume a larger increase in damping and natural periods.

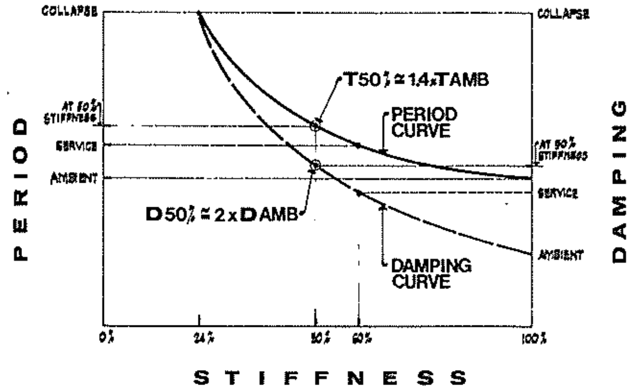

Fig. 2(c)—Damping and fundamental period as function of load level, causing deterioration of stiffness

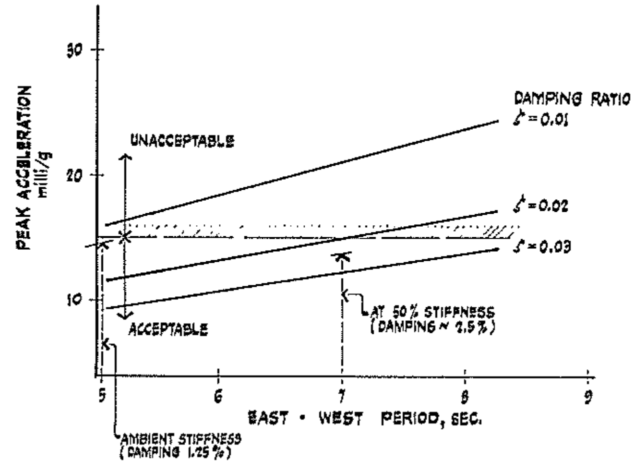

Fig. 2(d)—Dependence of accelerations on building periods and damping ratios

Fig. 2(c)* describes the relationship between the damping and the fundamental period as a function of the load level causing degradation in the stiffness of a structure. Notice that at service seismic loads, ambient stiffness was reduced by 40 percent. At 50 percent of ambient stiffness, the period T was increased by about 40 percent and the damping was double that observed at the ambient state. Fig. 2(c) does not provide absolute values as they change depending on the structure, the nonstructural elements, the quantity of the reinforcing, the source of the lateral loads, etc. For wind design, a common damping range of 1.25 to 2.5 percent is probably appropriate, with the larger value used to estimate the design forces and some intermediate lower value used to estimate the accelerations during a 6 to 10 year return period.

After the dynamic properties have been initially estimated (or later field-verified) the high-frequency force balance measurements can be used to evaluate the perception of motion at various stages of wind action for a wide range of possible building periods and damping ratios. The structure discussed, having a measured ambient period (after completion) of about 5 sec with 1.25 percent ambient damping, was projected conservatively to be only 50 percent as stiff at the larger design wind levels. The period at design wind levels was projected to be increased by about 40 percent and the damping level to double (i.e., 2.5 percent). Notice that for this concrete structure [Fig. 2(d)] the acceleration level of 15 milliG, the target for apartment structures for a 10-yr return, is not expected to be exceeded at any of the possible wind load, damping and period combinations.

EVALUATING ACCELERATION ESTIMATES

As a preliminary rule of thumb, assume for tall and slender structures that either increasing the mass or the stiffness will reduce the acceleration, roughly proportional to the square root of the ratio of initial mass to final mass, or the ratio of initial stiffness to final stiffness. Concrete structures, more readily than steel structures, can often utilize the increase in mass to also increase the stiffness, thereby providing a double benefit. For this reason the future of concrete for megastructures looks bright, especially now that extremely high concrete strengths are available.

As has been discussed, it is quite a trick — a balancing act — to produce a serviceable tall, slender structure. The owner for whom the structure is being developed must be involved in determining how much extra cost in increasing mass and stiffness should be considered so that a desired comfort level for the future occupant is attained. Involving the developer in this decision may be controversial, but it should not be viewed any differently than consulting with the owner about supplying more or fewer amenities such as light, heat, beautiful landscaping, etc.

Concrete is a more suitable material than steel for tall, slender structures because of its inherent larger damping and extra mass. As concrete structures are constructed taller and more slender, the structural shape and scale that will cross the threshold, the edge beyond which the control of motion perception is impossible to achieve or is economically unattainable, will eventually be reached. This threshold is already exceeded in some steel structures. It is obvious that the slenderness ratio is a major parameter determining this threshold, this new edge, for concrete structures, beyond which artificial means to increase damping to combat the problem of motion perception will become necessary.

DAMPING THE CONCRETE STRUCTURES ARTIFICIALLY

The net result of increased damping is a reduction of the dynamic response of the structure.[11] A larger damping will stop the building motion quickly and will not allow an inappropriate amount of time for wind (or seismic) forces to interact and increase the inertial forces (the cause of a large portion of the motion in the

*Fig. 2(c) is a plot constructed from preliminary information (provided by Professor Bertero of the University of California at Berkeley) the result of a US-Japan joint study of a large-scale three-dimensional model.

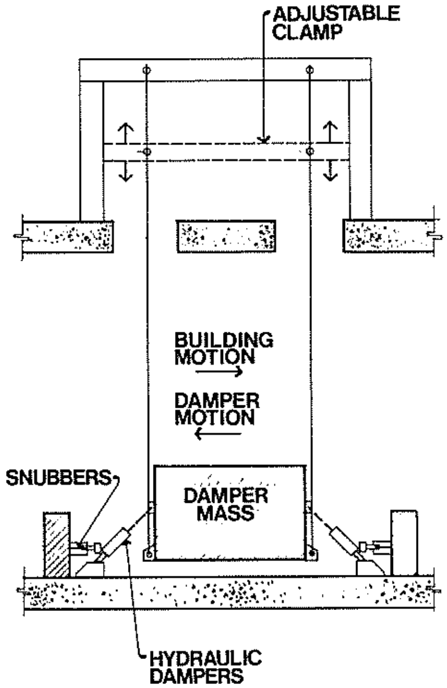

Fig. 3—Pendulum-type damper (Courtesy TACET Engineering Ltd., Toronto)

structure). Since concrete has a natural damping close to double that of steel, concrete is more suitable for constructing tall, slender structures. A concrete structure's extra weight also provides added hold-down loads to enhance stability. As already stated, the options to produce tall, slender structures that are nonsensitive to motion (but economical) by increasing the structure's stiffness and/or mass are limited. In evaluating each individual structure, there is a threshold, an edge, beyond which putting more stiffness or mass has a diminishing economic return; thus, other options must be reviewed. Such options may include reshaping the footprint, the height, or even the location, of the structure. Where these options are not feasible, introducing an artificial means of damping into a building must be considered.

A review of all possible dampers for about half a dozen of our recent tall, slender concrete structures eventually concentrated on a single application that has not been used extensively to date. A pendulum-type damper (Fig. 3) was selected as a possible economical solution for structures which may require an artificial increase to their natural damping. (It should not require much maintenance and would resist wind action because it is freely activated and is always out of phase with, and always opposes, the building's motion.) The space allocation required for a pendulum-type damping system is dependent on the anticipated natural period of the structure, its mass, and the required enhancement to the structure's natural damping.

A modal (generalized) mass of the structure (reflecting the kinetic energy involved) is computed using the following equation

$$\text{modal mass} = \sum_{i=1}^{i=r} M_i \, (\Delta_i/\Delta_r)^2 \tag{3}$$

where M_i = mass at level i, Δ_i = drift of level i, and Δ_r = drift of the roof level.

A small percentage of this generalized mass (between 1 and 3 percent) is then used to determine the damper's mass. The material used for the damper could be concrete or heavier, less space-consuming steel or lead. The travel distance of the damper, the physical dimensions required to provide space for the hydraulic cylinders and space for the safeguards (snubbers), will then establish the size of the required room. A 30 ft square room can usually accommodate the damper. A space must also be provided to allow for the cable movements, above the damper room. The fundamental period of the structure will establish the length of the pendulum, which to be efficient should have a period closely matching that of the structure. The fundamental period of a free, flexible cable-hung pendulum-type damper is independent of its mass and is a function only of the square root of the ratio of the length L, divided by the Earth's gravity

$$T = 2\pi \sqrt{\frac{L}{G}} \tag{4}$$

To accommodate uncertainties in determining the natural period, and to allow for variations of the natural period with time of service, the length of the pendulum can be adjusted with a moveable clamp. A building's period is expected to lengthen with time of service as more and more nonstructural elements such as partitions are disengaged from the structure, and as more and more cracks develop due to storms. Therefore, it will occasionally be necessary to adjust the free length of the pendulum. For this reason the damper's support should be located to permit a maximum length associated with the largest period possible.

The cost of implementing a damper must include the cost of the space allocated to house it, as well as the cost of installation. It is therefore useful to instrument and measure the structure's behavior, as it nears completion, to better predict whether there will be a need to install the damper immediately (and at lower cost) during the construction period, rather than at a later date. Such measurements can be taken (as already reviewed) by using a climbing crane to cause the structure to vibrate. The small, ambient motions caused by the crane are recorded and the structure's low-amplitude damping and natural periods thus obtained. From these readings, the design team can better project how the structure may behave during its service life. The decision to include a damper immediately, or to provide only a space and capacity for future installation, or to do none of the above can then be made prior to completion of the structure. The extra load for the damper must be accommodated in the design of the foundations and columns. This is a small penalty to pay for peace of mind.

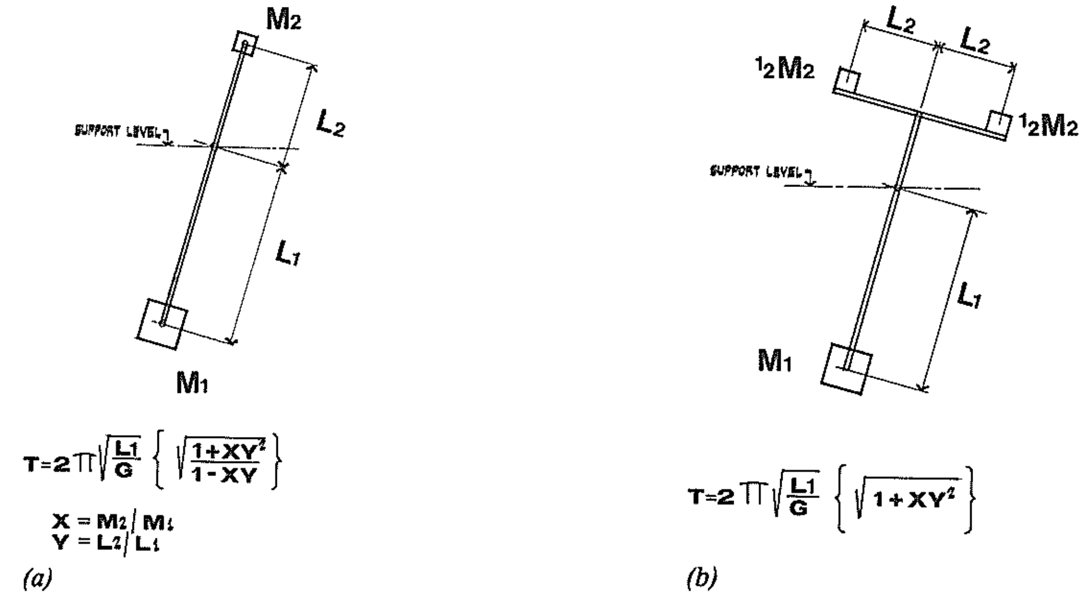

Fig. 4—(a) Rigid pendulum-type damper with counterweights; (b) flat-arm pendulum-type damper

While a freely suspended pendulum-type damper might be the most economical to install and operate, it is possible that height limitations may not allow full compatibility between the damper's and the structure's periods. It is then necessary to increase the damper weight and to increase the damping of the hydraulic cylinders to compensate for some loss of efficiency.

Other possible avenues to increase the damper's period without elongating its length will now be presented. The practical application of such dampers must first be analyzed and tested in a laboratory. They are mentioned at this embryonic state only to arouse the profession's interest and to indicate that there are numerous means possible to dampen a structure. One such option is to support the mass on somewhat flexible legs, which will introduce the effect of the supporting system itself to help elongate the period of the damper. Alternately, the supports of the pendulum can be made to slide slightly and be guided by adjustable springs. Another possible avenue (much more difficult to implement when the mass is large) is to make the pendulum rigid [Fig. 4(a)] and adjust its period by means of counterweights above the point of support. When the pendulum is rigid and supports loads at both ends, the period becomes dependent on the masses involved. Unfortunately, the effectiveness of the damper's mass is reduced by the counterweight. An offspring of this type of damper would be to flatten the arm above the support. [Fig. 4(b)] to accommodate even more stringent height limitations (this will not reduce the effectiveness of the damper's mass). A drawback of the last two options is that it would be necessary to develop an almost friction-free support to allow pivoting of the rigid pendulum. For this reason, these systems might be more appropriate for applications other than for massive structures. All of the options just stated will allow the damper's period to increase without elongating its length.

SUMMARY

Fundamental period computations and predictions for the service life of a structure will indicate a range of periods from a probable low to a probable high. From these, a slender structure's needs will be evaluated. The high-frequency force balance wind-tunnel model can be effectively used to predict results for this range of periods for several levels of damping. The maximum lateral loads for a 50 or 100 year return period are obtained in order to design the structure for strength and serviceability. The anticipated peak accelerations for a 6 or 10 year return period are also computed to evaluate the structure's range of perception of motion. This information can be tabulated to show predicted accelerations for a range of mean wind velocities acting on the structure from different directions, as well as to show the frequency of occurrence of such wind speeds. The engineers and developers can then evaluate the data to determine whether a damper is needed. Until confidence is gained concerning the reliability of dampers, it is not recommended that the lateral loads be reduced because of the damper's presence.

Concrete structures with slenderness ratios of 10 that have already been built are without dampers. The design process provided sufficient stiffness and mass to indicate only marginally a possible future need. Should the future dictate a need, some of these structures have been provided with the capacity to accommodate installation of a pendulum-type damper. Field measurements and/or observations of the structures already completed have not indicated to date that the decisons to omit the dampers were incorrect.

CASE STUDIES OF TWO TALL SLENDER MEGASTRUCTURES

The tall, slender megastructure in the foreground of Fig. 5 is Metropolitan Tower. Completed in 1985, with a triangular-shaped footprint, it provided the firm of

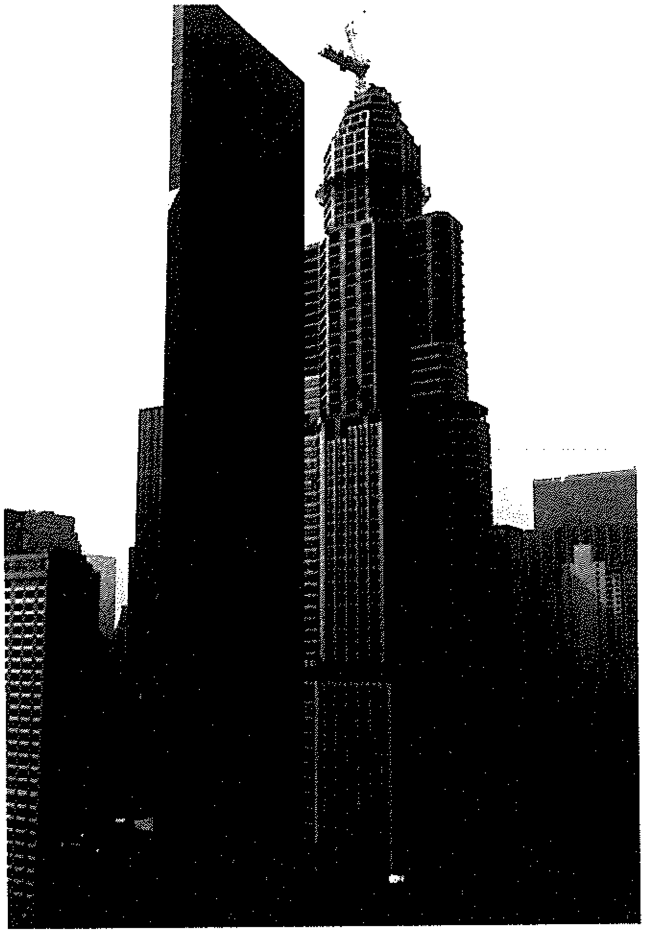

Fig. 5—*City Spire and Metropolitan Tower, north view (Photo: Berenstein Assoc.)*

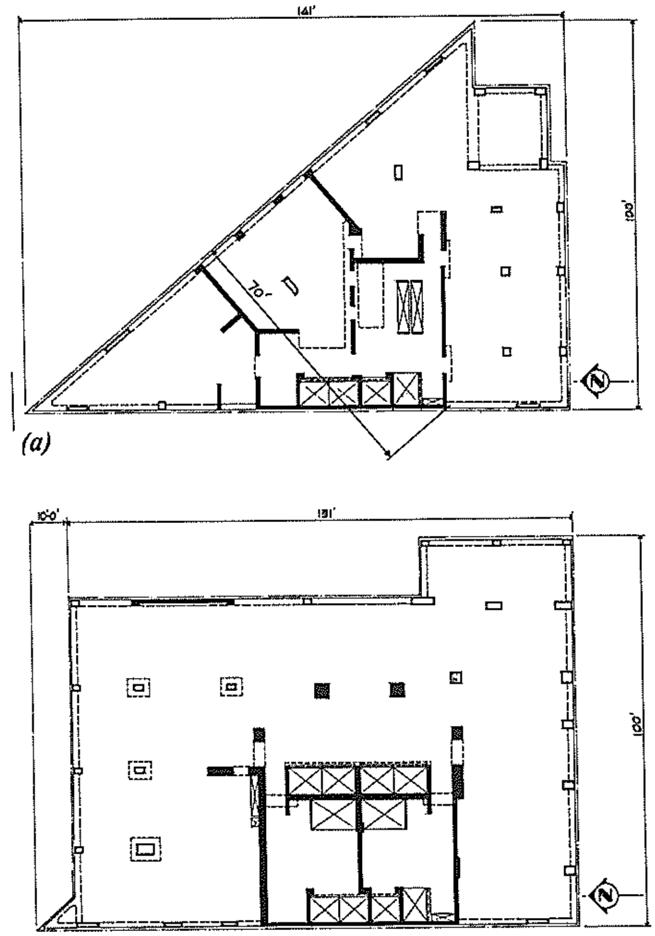

Fig. 6—Metropolitan Tower: (a) typical office level; (b) typical residential level

Robert Rosenwasser Associates, P.C., with a "new edge" that required studying the possible introduction of a damper for the first time.

Metropolitan Tower, a 716 ft tall, 68-story structure, is located at 146 West 57th St., New York City. The depth of the meandering shearwall (the main structural support of the triangular footprint) is about 70 ft [Fig. 6(a)]. Of the three available faces, the west face was a lot-line face, and therefore a place to accommodate the elevator shafts for the high-rise structure. It was recognized, and later verified in a wind-tunnel test, that the structure would support larger wind forces acting perpendicular to the hypotenuse of the triangle. Vortex shedding, which usually produces larger forces transverse to the wind direction, did not materialize for this structure because of its triangular footprint. Shearwalls [shaded in Fig. 6(a)] then migrate from the west lot line, meandering alongside apartment lobby and corridors, to the hypotenuse side of the triangle where additional columns were engaged via vierendeel action of the spandrel beams. Other frame elements, 20 in. deep spandrel beams along the periphery and 8.5-in. slabs at the interior of the structure, were needed to help counter large torsional loads since it was impossible to minimize torsional forces for all possible wind directions. This slender tower was somewhat stiffened by a wider base below the eighteenth floor [Fig. 6(b)]. However, part of the shearwall, and many of the columns, had to be transferred utilizing deep concrete girders at this level. These deep girders were utilized, via outrigger action, to engage additional supports to help divert hold-down loads for the shearwall and to equalize the strain in the supports.

This building (with 650,000 ft^2 of floor area that required approximately 30,000 yd^3 of concrete and 3600 tons of reinforcing steel) was designed for strength and to limit drift to acceptable levels to prevent nonstructural cladding and partition damage. Ample soft joints were called for at each level for both the exterior and interior nonsructural elements. The question of perception of motion was resolved by providing the capacity in the columns and foundation to install a damper.

Using three accelerometers, field measurements were taken when the structure reached its fifty-fourth floor and, later on, at its sixty-sixth floor (at the last possible date, allowing time for a "go/no go" decision with regard to installation of a damper), indicating that a damper was not needed. The extra cost to the owner was in providing a double design layout, with and without the damper. No material, except to support the damper's weight in the foundation and columns, was actually expended in the structure. This structure could

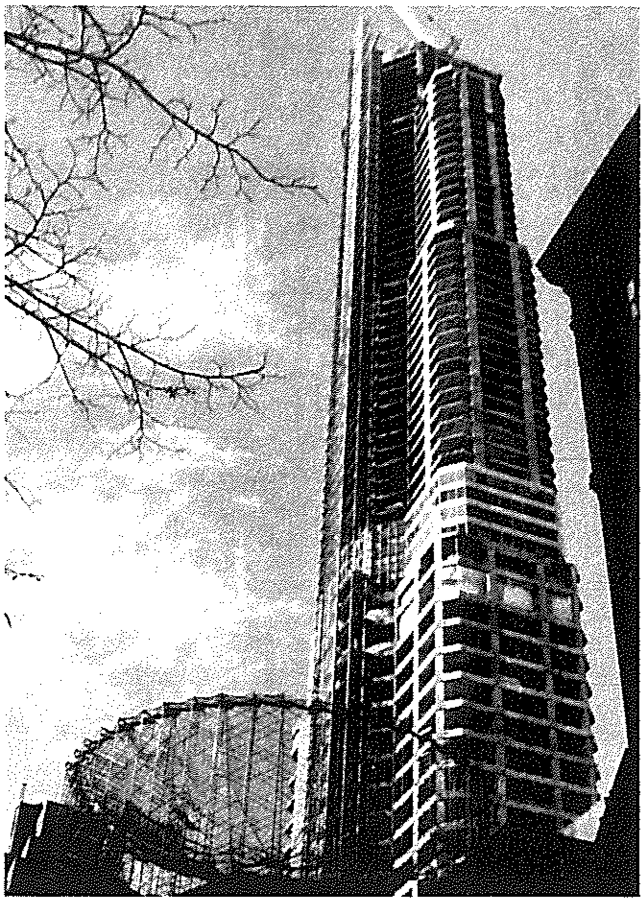

Fig. 7—City Spire (recipient of the Concrete Industry Board's 1987 Residential Award): east elevation (photo: W. Grossman)

accommodate a future damper, if found necessary during its service life, with some minor modifications and rerouting of some mechanical pipes.

City Spire, 156 West 56th St. (see Fig. 5 [background] and Fig. 7), displaced Metropolitan Tower as the tallest concrete structure in New York City — concrete placement reached the 800 ft level and aluminum-dome fins extended the height to 814 ft above grade. When completed, it was the second tallest concrete structure and, at a 10-to-1 ratio is the tallest, most slender structure (concrete or steel) in the world today. The structure occupies about 830,000 ft^2, and required 43,000 yd^3 of concrete and 4700 tons of reinforcing bars for its 77 construction levels (including mechanical and below-grade levels). It was topped in April 1987.

The critical wind direction for this building is from the west, which produces maximum across wind action in the short (north-south) direction. Wind-tunnel studies indicated possible resonance with wind forces, which could cause vortex shedding to interlock with the structure and increase wind loading, and also the discomfort level. This possibility was eliminated by adding some stiffness and mass to the structure.

The modeling of City Spire was complex because the structure is subdivided into nine major structural subsystems with many setbacks. Fig. 8 displays a few of

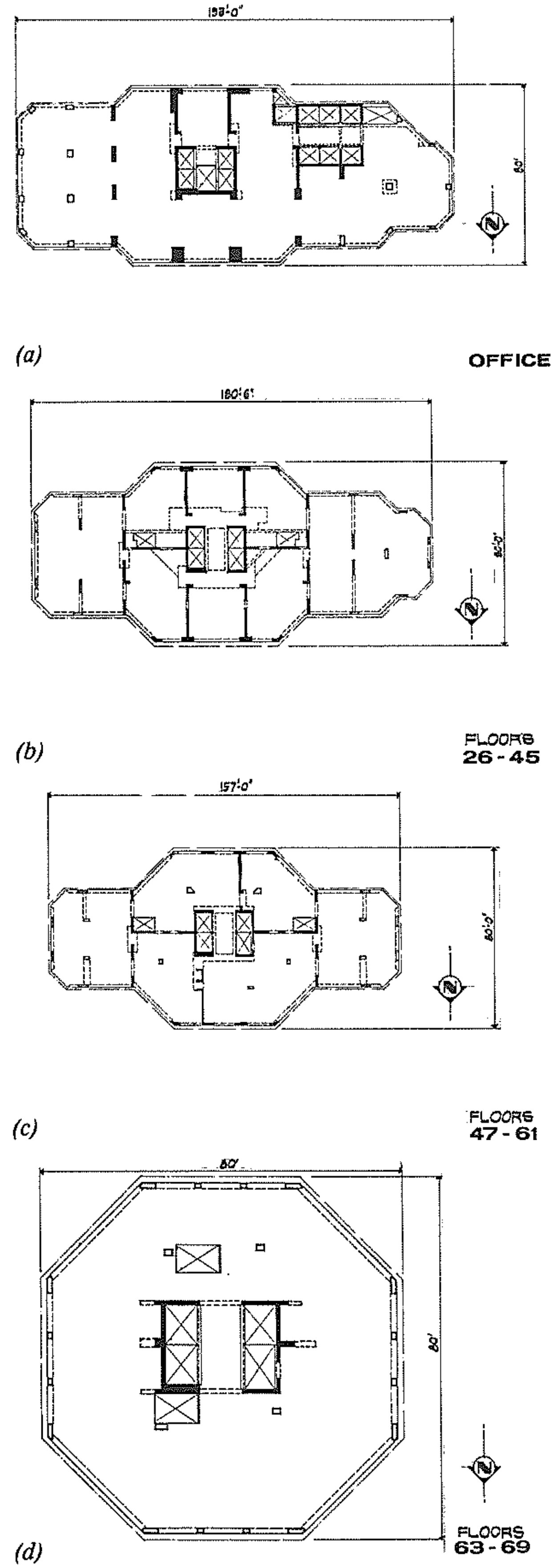

Fig. 8—City Spire: (a) typical office level; (b) typical lower residential level; (c) typical midrise residential level; and (d) typical octagon level

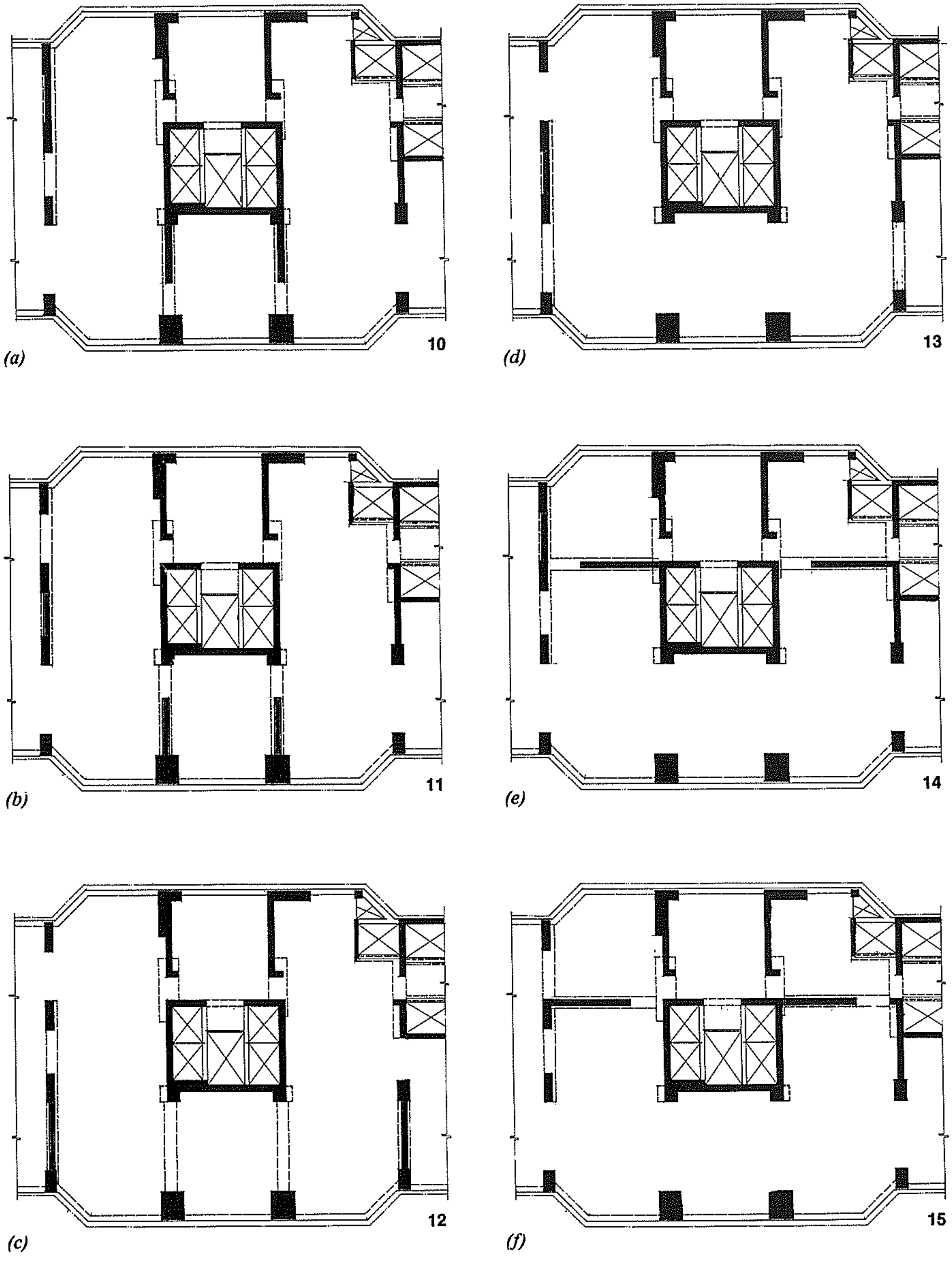

Fig. 9—Shearwall open tube connecting elements: (a) tenth floor; (b) eleventh floor; (c) twelfth floor; (d) thirteenth floor; (e) fourteen floor; (f) fifteenth floor

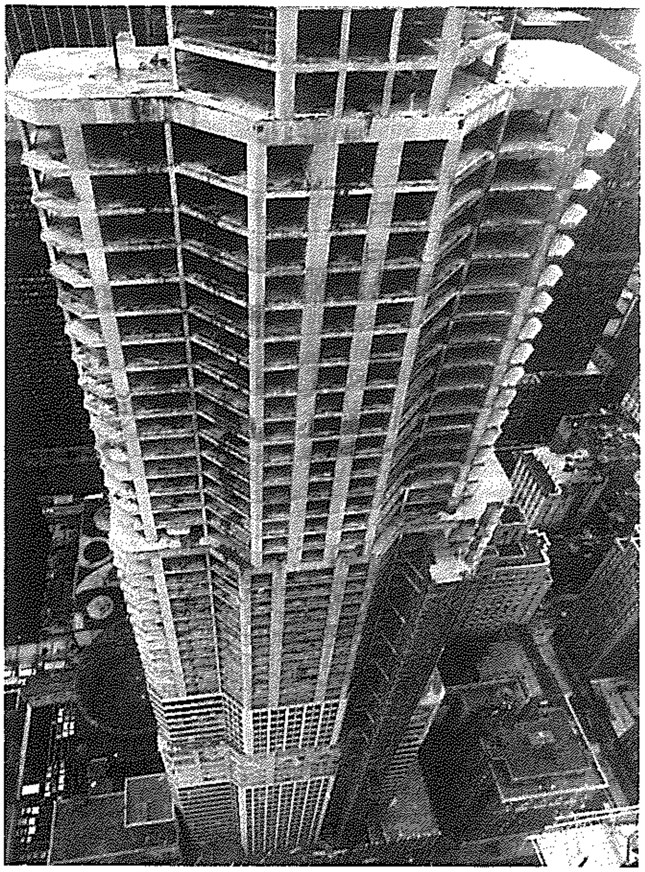

Fig. 10—City Spire, construction view, north elevation, showing setbacks and available Manhattan views (photo: Berenstein Associates)

these layouts. The computer program TOWER was used 15 different times to fine tune the project and to help accommodate last-minute revisions (some implemented after the construction started). Finite element analysis (using SAPIV, which required the solution of over 13,000 simultaneous equations) was also run as backup to the calculations made by TOWER.

The main structural system is a "shearwall/open tube" (shaded members in the figures), which traverses the center (80 ft wide) octagon in each direction. Coupling beams were used at the residential levels [Fig. 8(b) through (d)] and internal space diagonals with coupling beams were used in the lower office levels [Fig. 8(a) and 9(a) through 9 (f)] to connect the many parts in this system. Staggered rectangular concrete panels were used to form the space diagonals. These panels occurred on a few preselected office levels to provide continuity between the 5.5 x 7 ft jumbo columns located on the north octagon face and the center residential-elevator core. The east and west octagon columns were similarly connected by staggered concrete panels. The available open office floor space was only somewhat reduced by these panels. Access routes in both the E-W and N-S directions, and open panoramic views (essential for Manhattan occupany) for the office levels and the residential levels (Fig. 10), were thus provided. The apartment levels above the twenty-sixth floor [see Fig. 8(b)] provided easier N-S and E-W connections along demising partitions which required numerous coupling beams to connect the many parts of the shearwall. Above the sixty-second floor [Fig. 8(d)] the wings, which extend from the 80 ft octagon center, were eliminated. A large open span, free of supports, was maintained between the exterior and center elevator core to accommodate flexible duplex and penthouse layouts.

The design problems were further complicated by the intrusion of a City Center Theatre stage extension into the structure and providing (for the octagon north and south faces) glass-cleaning access from the structure's interior. Any concrete columns at the periphery could not be too wide (to allow for hand-reached cleaning distance) and the beams not too deep (so that views are not obstructed). Some of the structural solutions to such restrictions, especially at the many setbacks requiring large column load transfers, are evident in Fig. 10. At each of the setbacks, transfer girders joined other belt beams to mobilize (via vierendeel and outrigger action) additional supports and tie down loads to the shearwall-open tube lateral resisting system.

The columns' concrete strengths in both structures varied between 8300 and 5600 psi. Corresponding strength of the floor members, respecting the allowable ratio of 1.4 of the ACI Building Code[12] varied between 5950 and 4000 psi. The prefabricated cladding for these structures made special handling of the construction process necessary to compensate for the elastic and initial long-term (creep and shrinkage) shortening of the concrete supports. The concrete contractor was instructed to build in an extra ⅛ in. per floor to compensate for initial construction losses. Ample soft joints were provided for future losses including racking and temperature demands.

In the author's opinion, these two megastructures represent the "new edge" of our "sphere of knowledge" — our understanding of high-rise concrete construction. It was hoped to postpone involvement with even more slender structures for a while, but this has not been the case.[13] With the passage of time and service, the structures will provide us with the knowledge necessary to progress to even more esoteric scales while they, in turn, move (hopefully successfully) toward the center of the "inner core" of our understanding about structures. When this time comes, the engineering profession is expected to face even newer "edges" to test its ability to satisfy mankind's dreams and aspirations, daring as they may be.

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Credits for the slender structures discussed are: Morgan Court: owner — M. Perlbinder Realty Corp., architect — Liebman and Liebman Assoc., P.C., structural engineer — Robert Rosenwasser Assoc., P.C., general contractor — Perlbinder Construction Co., concrete contractor — North Berry Concrete Corp.; and for Metropolitan Tower: owner — H. Macklowe Real Estate Company, architect — Schuman, Lichtenstein, Claman and Efron with design input from Macklow/Derman/Werdiger, structural engineer — Robert

Rosenwasser Assoc., P.C., construction manager — HRH Construction Corp., and concrete contractor — North Berry Concrete Corp., and for City Spire: owner — Eichner Properties, architect — Murphy/Jahn, structural engineer — Robert Rosenwasser Assoc., P.C., construction manager — Tishman Construction Corp., and concrete contractor — S & A Structures, Inc.

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CONVERSION FACTORS

1 ft = 0.305 m
1 in. = 25.4 mm
1 ft^2 = 0.0929 m^2
1 lb/ft^2 = 4.882 kg/m^2
1 ksi = 6.895 MPa
1 psi = 0.006895 MPa
1 yd^3 = 0.765 m^3

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