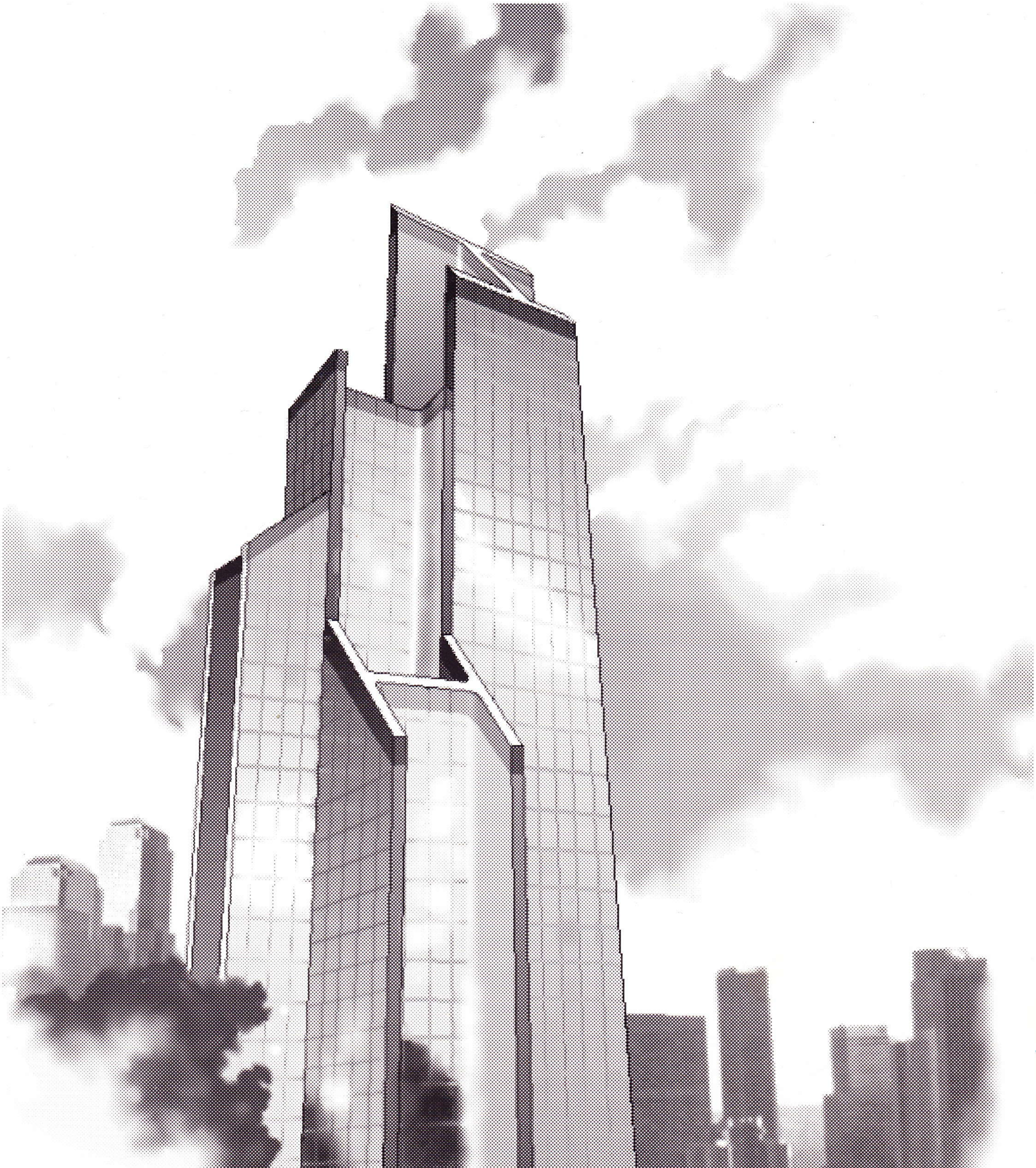
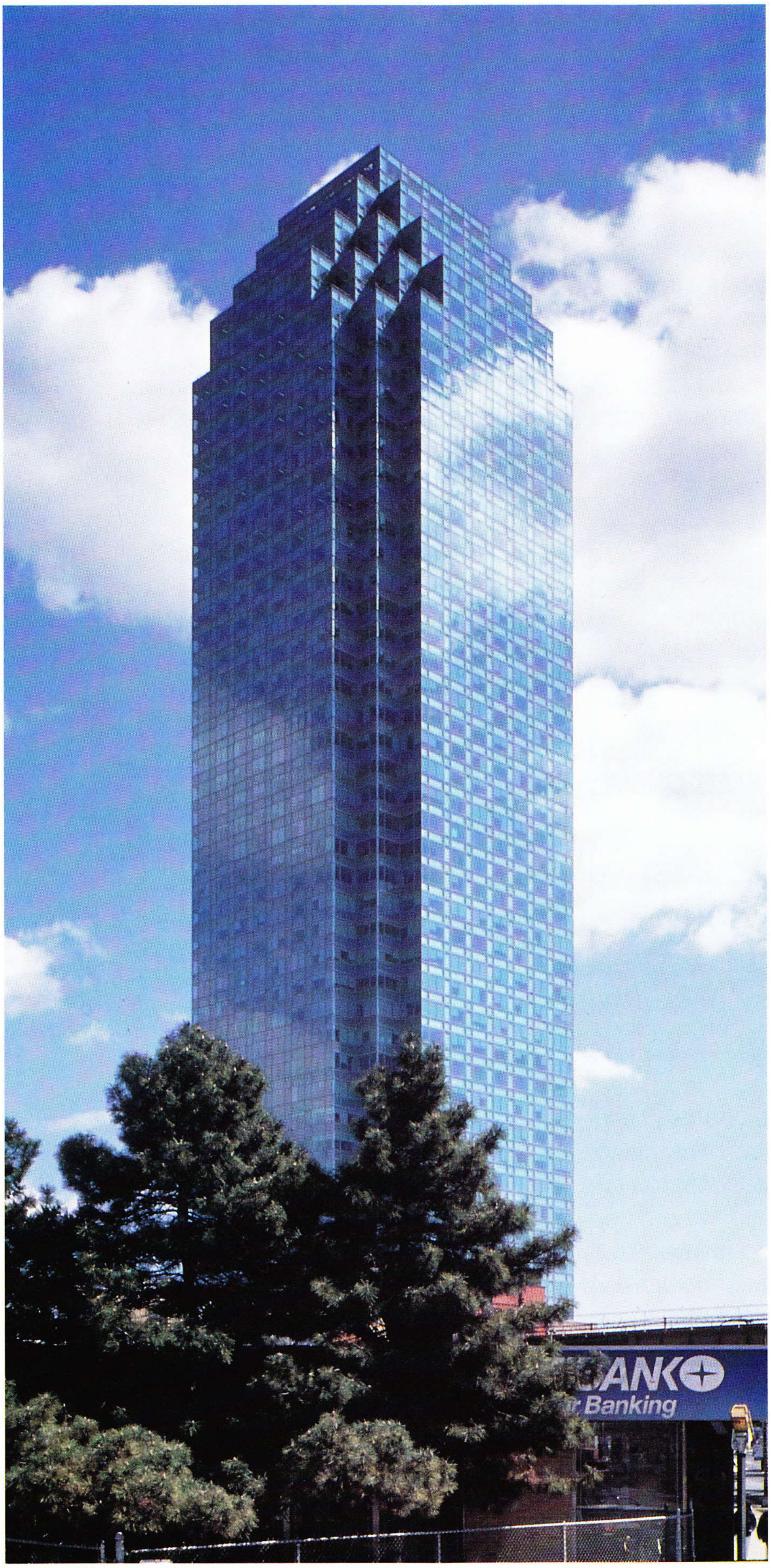
EUROPROFIL



High-Rise Buildings



In the last decades, significant developments in architectural expression and the increasing demand for lighter and taller buildings resulted in a systematical evolution of structural systems. Steel structures were predominant in these developments and, profiting from the inherent properties of this material, new steel framing systems emerged.

The developments in the steel industry contributed to the structural efficiency of these new framing concepts by providing new heavy sections and high-strength steels with outstanding mechanical properties. The floor framing systems for highrise buildings are generally identical to those used in medium-rise structures despite the need for larger spans and column transfer facilities at ground level for lobbies or street activities.

The main design criteria for tall buildings are governed by the lateral stiffness in order to resist wind and earthquake forces.

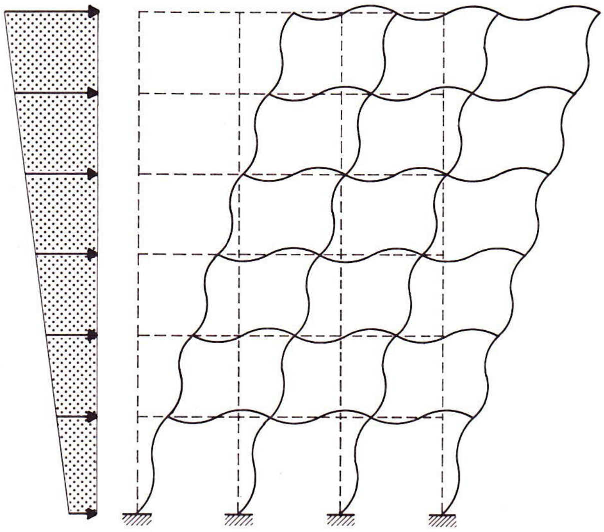
The following is a brief description of different structural systems related to the lateral stiffness in order to resist horizontal forces.

Citicorp at Court Square, New York, 1988
Skidmore, Owings & Merrill, New York

Shear Frame

Moment resistant beam to column connections create shear frames or Vierendeel frames which provide lateral stiffness in both orthogonal directions. The efficiency towards lateral stiffness is controlled by the individual stiffness of the members depending on the section and the length of girders and columns. The resistance to sway deflection is mainly governed by the bending of beams and columns due to wind forces and less from column shortening or cantilever action. The figure below shows the theoretical sway deflection of this framing system under the action of wind forces. Common bay dimensions range between 6 m to 9 m.

The shear frames can be located only in the exterior facade or, for more efficiency, in all vertical sections according to the column grid. This frame system has the advantage of large



Sway deflection under wind pressure

rectangular openings in the facade. However, the fabrication of moment resistant connections is expensive compared to other systems. Additionally, steel consumption is rather high and reduces the range of application to lowrise buildings.

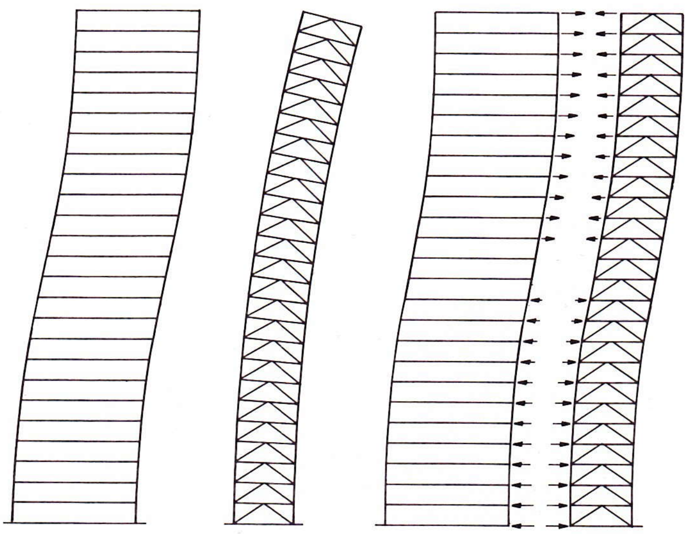
Shear truss-frame

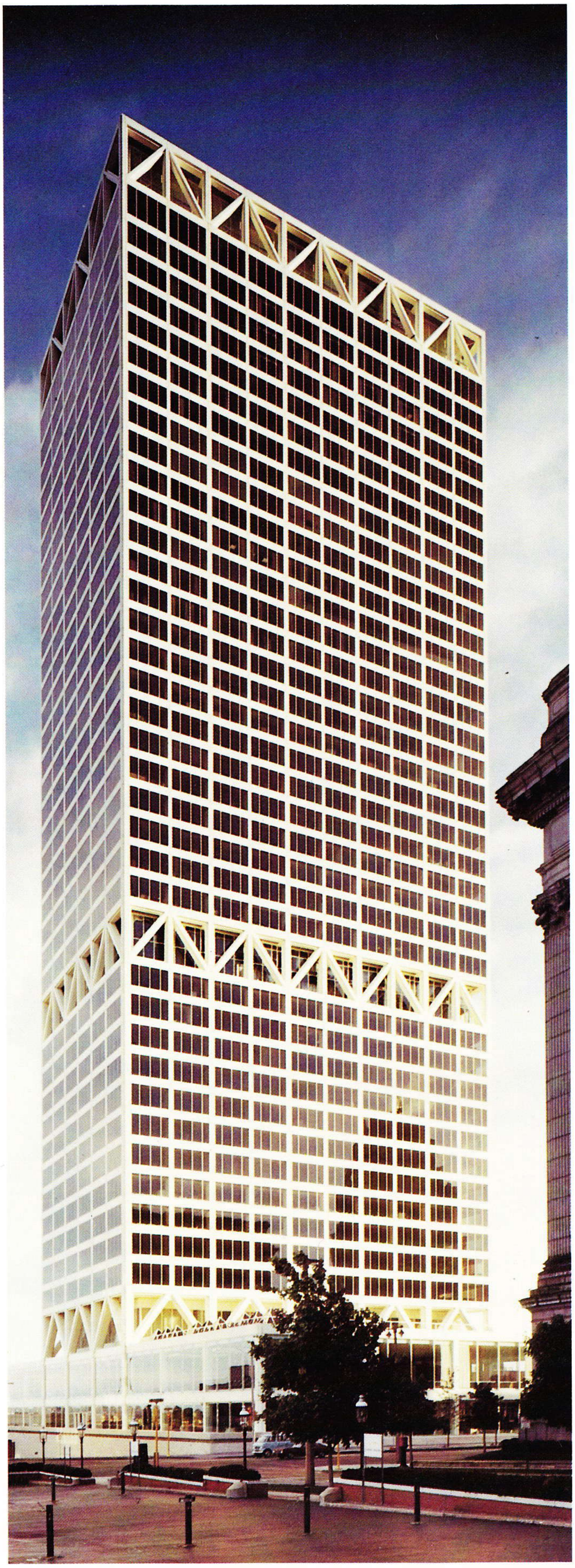
Vertical shear trusses located around the inner core in one or both directions can be combined with perimeter shear frames in the facade of a building. These frame-shear truss interacting systems are considered to be the most economical steel systems for buildings up to 40 storeys. In some cases the inner core can be executed using concrete shear walls, thus substituting the steel trusses in K, X or single brace form which may interfere with openings that provide access to, for example, elevators. The K-form is preferred for trusses since in the case of X and single brace form bracings the influence of gravity loads is very important. In seismic areas, eccentrical bracing is used in order to enhance the damping behaviour of the overall structure. The interaction of shear frames and vertical trusses produces a combination of two deflection curves with the effect of more efficient stiffness.

Outrigger and Belt Trusses

Another significant improvement of lateral stiffness can be obtained if the vertical truss and the perimeter shear frame are connected on one or more levels by a system of outrigger and belt trusses. The outrigger truss leads the wind forces of the core truss to the exterior columns providing cantilever behaviour of the total frame

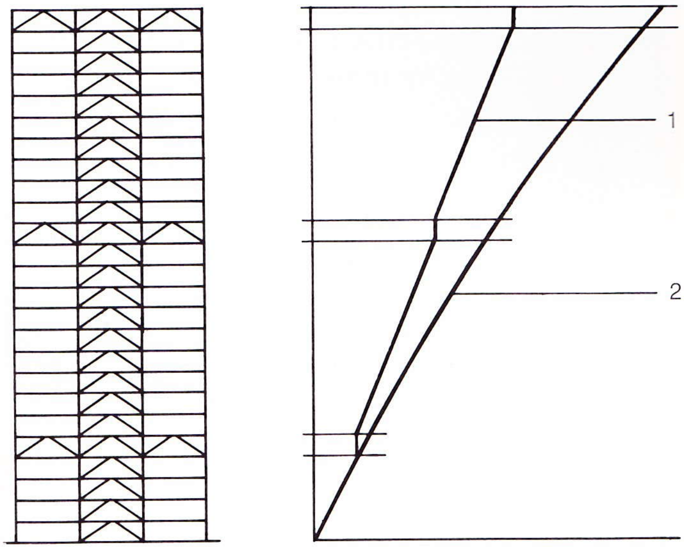
Frame-shear truss interaction





system. The belt truss in the facade improves the cantilever participation of the exterior frame and creates a three-dimensional frame behaviour. The overall stiffness can be increased up to 25%.

The efficiency of this system is related to the number of trussed levels and the depth of the truss. In some cases the outrigger and belt trusses have a depth of two or more floors. They are located in services floors where there are no requirements for wide open spaces. These trusses are often pleasingly integrated into the architectural conception of the facade.



Deflection curve
1 Drift with belt truss
2 Drift without belt truss

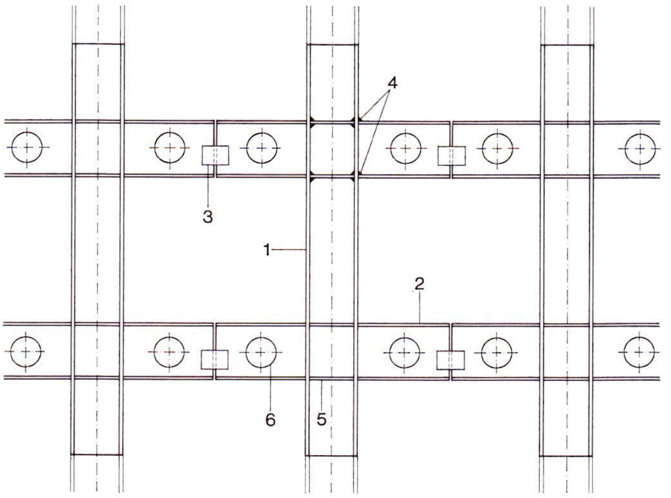
The Framed Tube

To achieve cantilever action of the exterior frame, the column spacings are reduced to 3 m – 5 m and are connected by moment resistant joints to the spandrel beams at the overall frame perimeter. Thus the part of sway deflection due to the shear frame can be reduced to less than 25%. Such framed tubes that provide only small openings for windows have a sway deflection behaviour similar to a hollow tube fixed at the ground.

First Wisconsin Center, Milwaukee, Wisconsin
Skidmore, Owings & Merrill, Chicago

The framed tube system calls for heavy beam and column sections with moment resistant joints. In order to avoid welding on site and to facilitate erection, column-trees are prefabricated in the workshop.

The spandrel beams are bolted on site at mid-span. This columntree system is very efficient for structures ranging between 30 and 100 storeys. The erection of 3 to 4 storeys a week is commonly achieved.
The framed tube, due to the close spacing of the columns, has a determinative influence on the architectural treatment of the facade. If wider column spacings are required and in case of a rectangular plan configuration the diagonalized tube system is often more suitable. This system



Column-tree system
1 column section
2 beam section
3 bolted connection
4 welded connection
5 continuity plate
6 penetrations for mechanical services

introduced in the John Hancock building in Chicago provides the tube action by an exterior diagonalized truss.
The trussed tube with diagonals on all four building facades emphasizes space frame action. The overall efficiency i.e. fabrication, erection and steel consumption advantages have to struggle with the need for a uniform and regular exterior architecture.

John Hancock Center, Chicago, Illinois, 1970
Skidmore, Owings & Merrill, Chicago

Developments in the Steel Industry

It is obvious that high-rise structures have to cope with enormous gravity loads compared to other structures. On the other hand, the overall stiffness of high-rise buildings under the action of lateral forces, such as wind and earthquakes, is governed by the stiffness of the individual components of the steel structure.

To increase the performance of hot rolled sections, two interacting properties have to be considered: stiffness depending on the geometrical section and its influence on deflection and buckling behaviour on the one hand and the steel strength controlling the load bearing capacity of beams and columns on the other hand. Furthermore, the mechanical properties as, for example, the weldability have a determinative influence on fabrication costs.
EUROPROFIL, one of the main suppliers of rolled sections recognized at a very early stage the challenge to meet these requirements.

Tailor-Made Beams

For the design of high-rise buildings, the structural engineer, demanding heavier sections as those specified in the american standard for rolled sections (ASTM A6), frequently specifies expensive built-up sections. To offer a valuable alternative, the concept of tailor made beams has been developed, enabling flange thicknesses of up to 140 mm. Some of these tailormade beams have been introduced in ASTM A6–88. Thanks to this new concept, rolled beams can be used where built-up sections were previously required.
Tailor-made beams are rolled sections produced with the standard sets of rolls, within the usual rolling cycles. Only the thickness of flange and web are adapted to the customer's needs. With reference to ASTM A6 where rolled H-shapes are denoted as "W" (plus nominal height in mm), the tailor-made beams are called "WTM". These heavy jumbo sections and also the W 920x420 series are successfully used for columns and spandrel beams in exterior moment frame structures.

Specimen of a heavy jumbo section, successfully used for both columns and beams

HISTAR – a New Generation of Rolled Beams

The new Histar-grades for rolled beams combine properties such as high yield strength up to 500 MPa, excellent toughness at low temperatures and outstanding weldability, considered incompatible until now. This was achieved by an innovative Quenching and Self-Tempering (QST) process which facilitates the cost-effective production of a new generation of beams.
Even under extreme conditions, Histar beams stand guarantors for space and weight saving design.

A Case for Steel

The case studies presented on the following pages will demonstrate the numerous advantages of structural steel in the design of modern high-rise buildings.

Architects
and Structural Engineers:
Skidmore, Owings & Merrill,
San Francisco

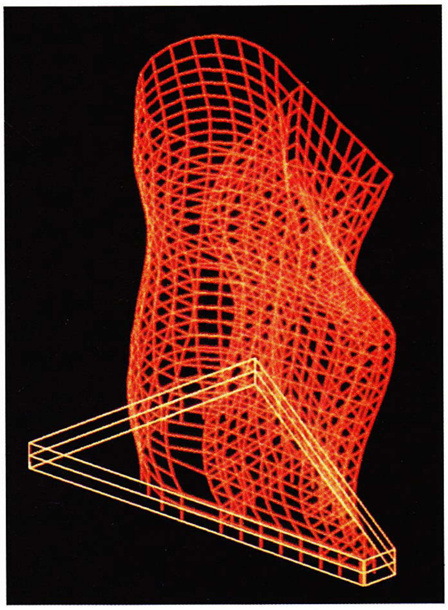
Developer:
Honorway Investment
Corporation

Site Description

The Building is shaped in plan like a tear drop. It rises from a triangular base that covers nearly the entire 2.100 m² site, and includes 15 floors of offices and 63 condominiums. Excavation of the bay mud site and the installation of a heavy internal shoring system was a tricky operation. The shape of the site created a prominent building where the centre of rigidity and mass are eccentrically placed.

Framing System

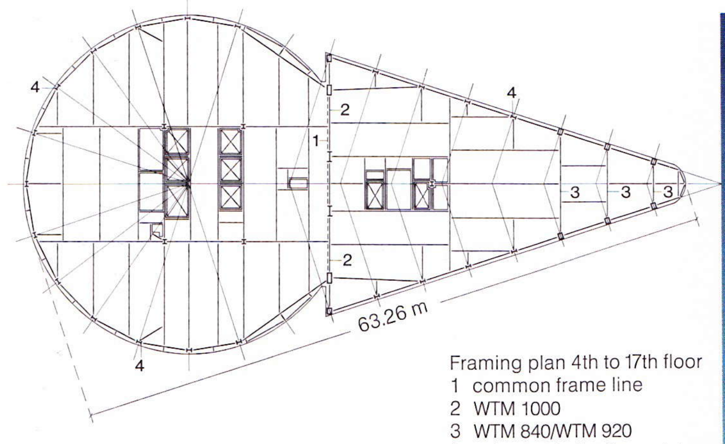
To create a structural system that could provide the building with enough stiffness to minimize its natural tendency toward wild lateral and torsional displacement under seismic and wind loads, the engineers developed a multiple tube framing system.



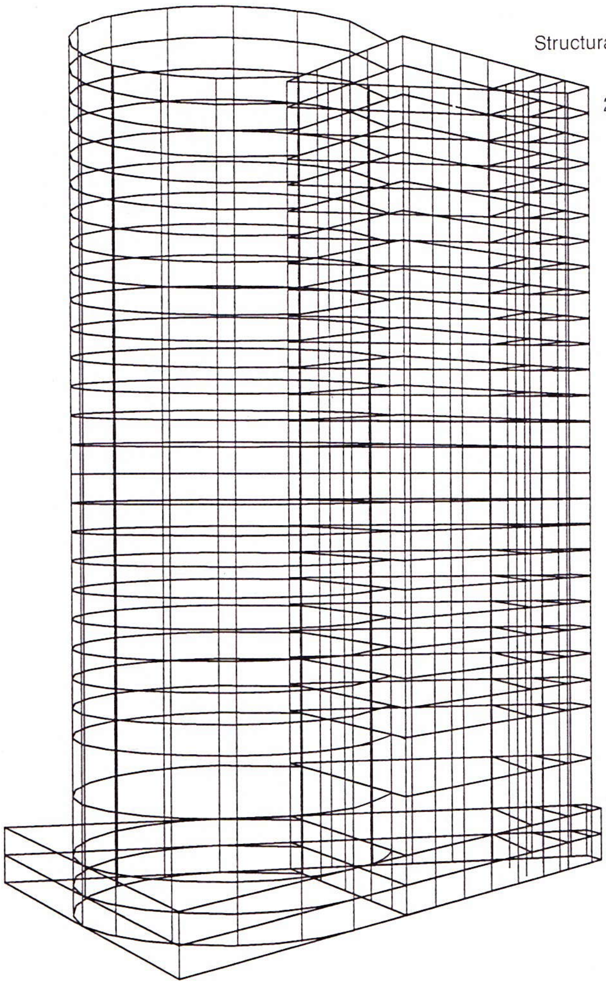
Computer simulation of lateral and torsional displacements under seismic and wind loads

In this project a perimeter ductile moment-resisting space frame is connected to a heavy central frame along the axis where the triangular and circular portions of the building meet. To strengthen the connection between triangular and circular portions, a 9,50 m long tangential beam was added, spanning the columns between the circular and triangular portions on floors 3, 18, 19, 20, 25 and 26. The central frame is the most important part of the structure, taking most of the shear forces. In fact, the building is designed to withstand seismic base shear forces 50% greater than those specified in the San Francisco building code, and to deflect only 230 mm each way. On each floor, the central frame has a row of six columns. The two interior rolled columns are flanked by two 1000 mm deep box columns, which are flanked in turn by two 640 mm deep box columns. Deep rolled girders span between the columns.

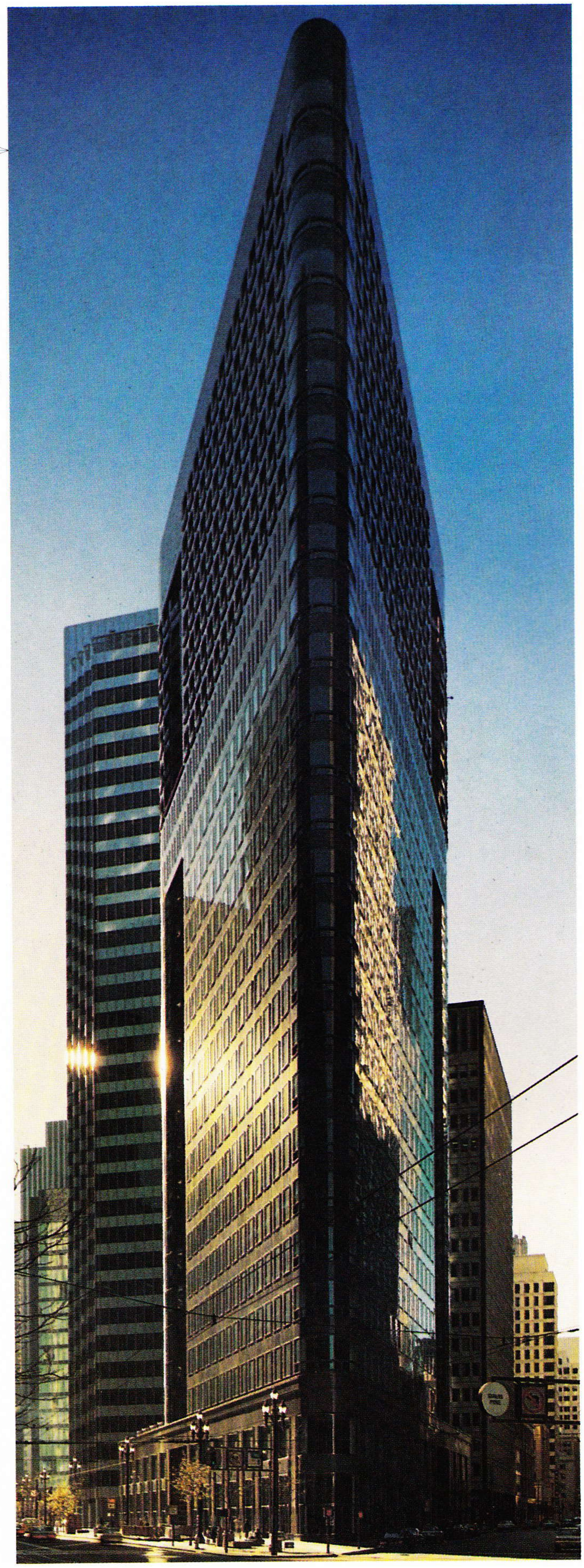
388 Market Street, San Francisco



Framing plan 4th to 17th floor
1 common frame line
2 WTM 1000
3 WTM 840/WTM 920
4 columns in the frame tubes consist of WTM sections
(WTM = tailor-made beams)



Structural scheme
26th floor
3rd floor
street level

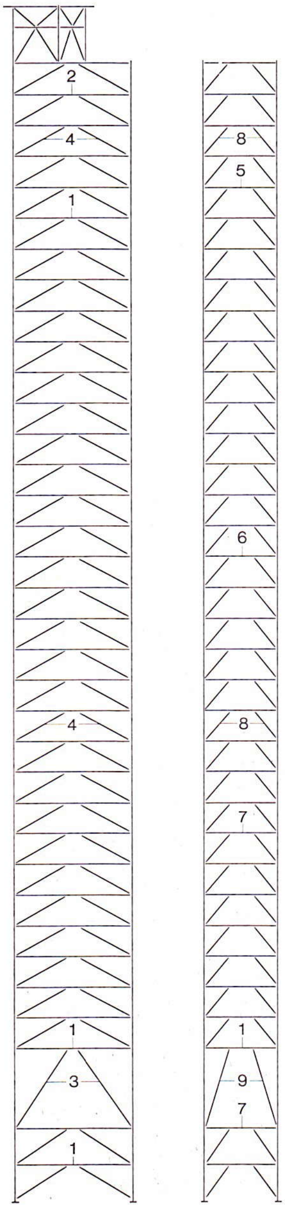


Massive, 1000 mm deep, rolled-shape steel columns and beams were specified for the central frame dividing the triangular and circular portions. To further strengthen the connection between the triangular and circular portions, a 9,45 m long tangential beam was added, spanning the columns between the circular and triangular portions on floors 3, 18, 19, 20, 25 and 26.

Architect:
John Burgee Architects
with Philip Johnson,
New York, N.Y.

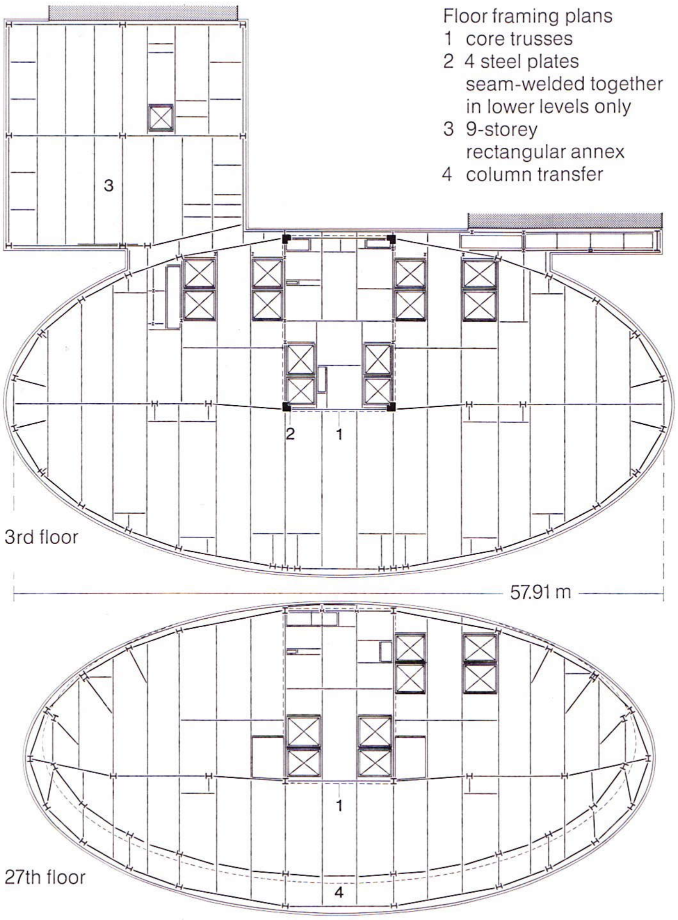
Structural Engineer:
The Office of
Irwin G. Cantor P.C.
New York, N.Y.

Developer:
Gerald D. Hines Interests,
Houston, Texas, and
Sterling Equities, Inc.,
New York, N.Y.



1 W 920 x 201 – W 920 x 446
2 W 840 x 193
3 W 360 x 382
4 ½ W 360 x 147 – ½ W 360 x 287
5 W 610 x 92 – W 610 x 113
6 W 690 x 125
7 W 760 x 147 – W 760 x 257
8 ½ W 360 x 448 – ½ W 360 x 134
9 ½ W 360 x 179

53rd at Third, New York



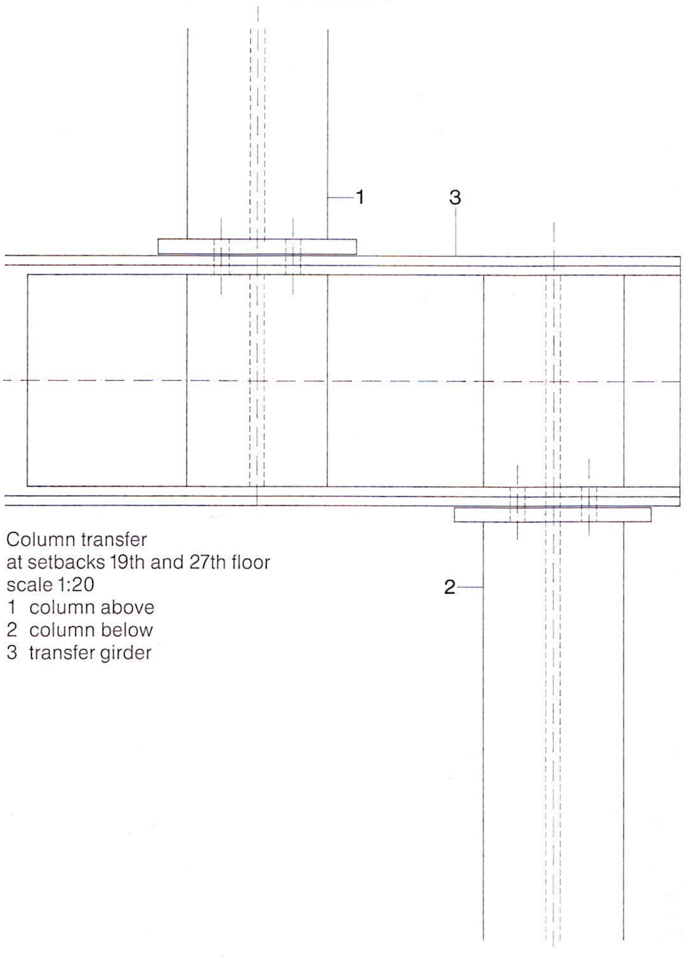
Floor framing plans
1 core trusses
2 4 steel plates seam-welded together in lower levels only
3 9-storey rectangular annex
4 column transfer

A unique architectural scheme

The elliptical office tower has an enormous skyline identity because of its shape and the striking contrast in the colour of the polished and flame-finished red granite alternating with the stainless steel bands and tinted grey glass. Setbacks at levels 19 and 27 enhance the building's sculptural form. The open and grand and welcoming lobby is two storeys high and ringed by 8,50 m tall columns. The building rises in three tiers of eighteen, eight and eigth storeys.

A telescopic framing system

Structural steel was selected as it was best equipped to handle the long spans as well as locate the wind system partially in the core and partially at the exterior. Four built-up jumbo steel columns in the core take 80% of the wind load and 50% of the gravity load on the building. To get the wind loads to the core, the engineer designed a moment frame with partially bolted, partially semi-rigid connections for the structure's perimeter girders. The wind load is transferred to the core through the floor diaphragms. The semi-rigid connections transfer half the gravity loads to the core and help the building resist its tendency to twist with the wind. The other half of the gravity load is carried to the foundation by the perimeter columns, which also resist torsion. Because the elliptically shaped frame left little margin for error, all moment connections were field-welded. Where the girders were joined to a column flange (in the semi-rigid moment connection), the girder's web was bolted to the column's shear plates through horizontal slots and then the flanges were field-welded. A loose top plate was butt-welded to the column flange and then fillet-welded to a plate that was shop-attached to the column. The welding was completed before the bolts were torqued into their final position.



Column transfer
at setbacks 19th and 27th floor
scale 1:20
1 column above
2 column below
3 transfer girder

Architects
and Structural Engineers:
Skidmore, Owings & Merrill,
New York, N.Y.

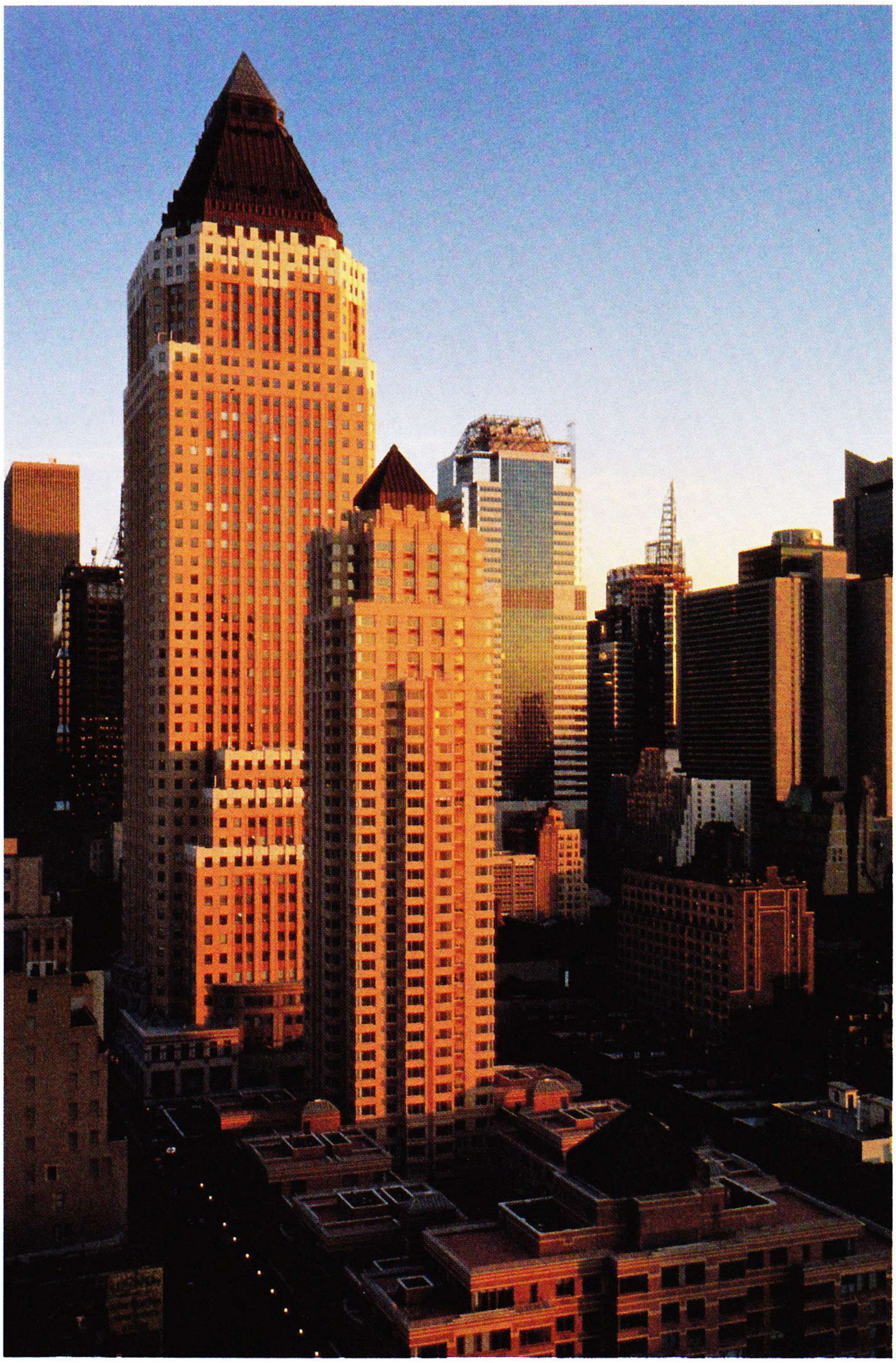
Developer:
New York Communications
Center Associates,
New York, N.Y.

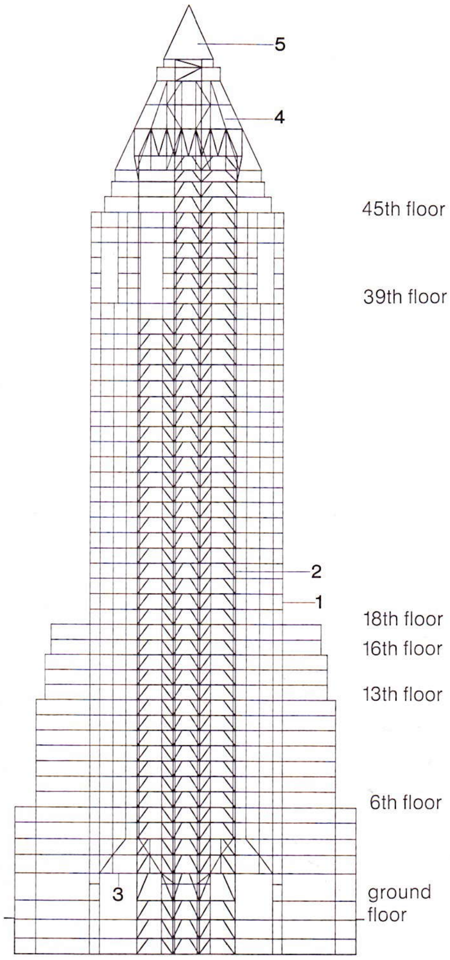
A new landmark for NYC-skyline

The 53-storey commercial tower is separated from the 41-storey residential tower by a multi-level plaza. The commercial building has four elliptical entrance lobbies, each set off by 10,60 m high granite-clad columns. The spandrels in the glazed brick exterior are set back to define the verticality of the shaft.

Exterior tube with tree-columns

The two-storey high columns were fabricated with four and sometimes two half-span spandrel beams welded to them to form "trees". These tree sections were shop-fabricated with 1,50 m stubs extending out either on both sides or on one side, depending on where the columns were to be placed. One advantage of the tower's framing being a perimeter tube framing system was that its stiffness facilitated some of the column

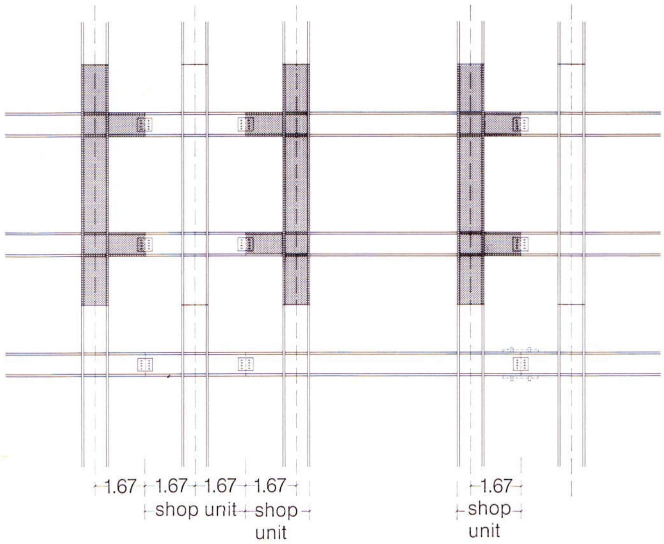




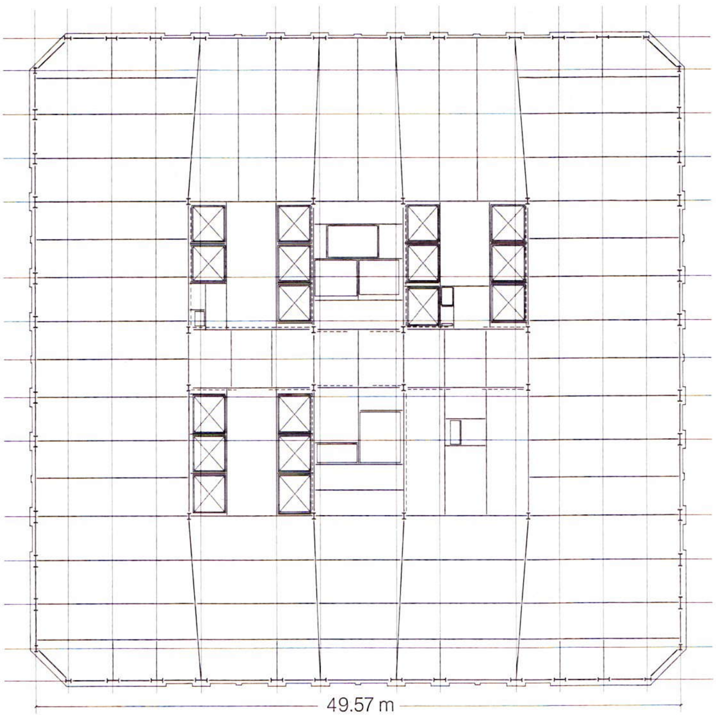
Structural frame elevation
1 exterior tube, moment connected columns and spandrel beams
2 braced frame, frames terminate as elevators drop off
3 tube transfers at ground floor arcade
4 braced roof frame supported on core
5 space frame supporting skylight

Worldwide Plaza, New York

Typical shop-fabricated erection units

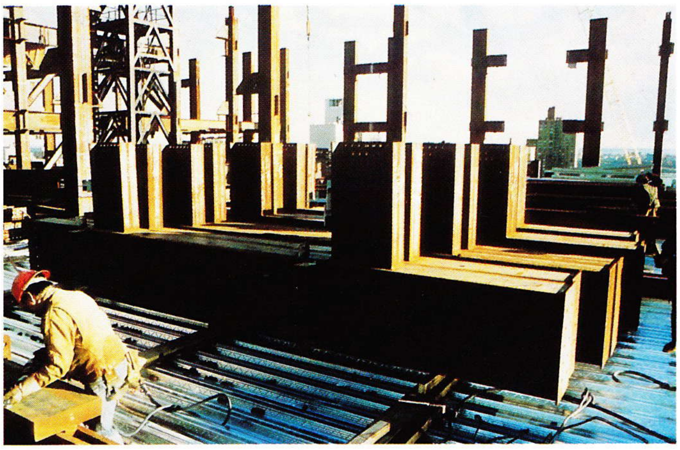


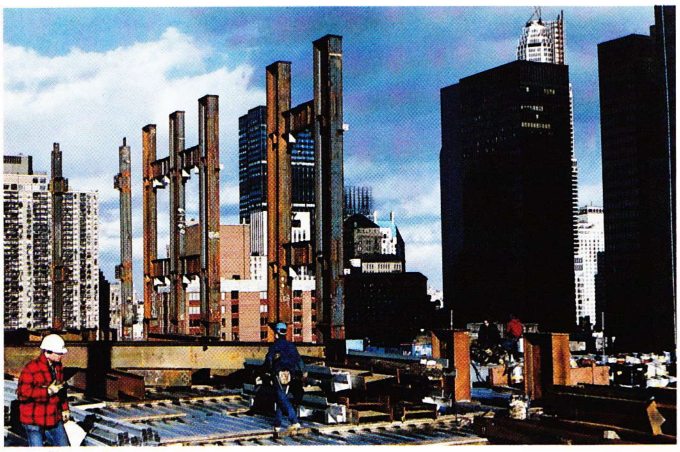
transfers taking place throughout. This building project is unusual because both the arcade at grade level and the copper-clad roof are open to wind forces on the interior walls. Some of the bracing stops at the 16th floor, some stops at the 40th and the rest continues to the 50th floor. The exterior tube resists most of the overturning moment and the braced core resists most of the shear forces. Tight bay spacing was needed at the corners to help transfer the wind loads around the corners of the building.



Framing plan 19th to 39th floor

The structural solution using irregular column spacing resulted in a unique expression for a tube-framed building and exemplifies the cooperation between structural and architectural disciplines.





Architects:
Johnson Fain and Pereira Associates

Structural Engineer:
Albert C. Martin & Associates,
Los Angeles

Developer:
JBM/Urban Development Co.,
Los Angeles

A highly articulated facade

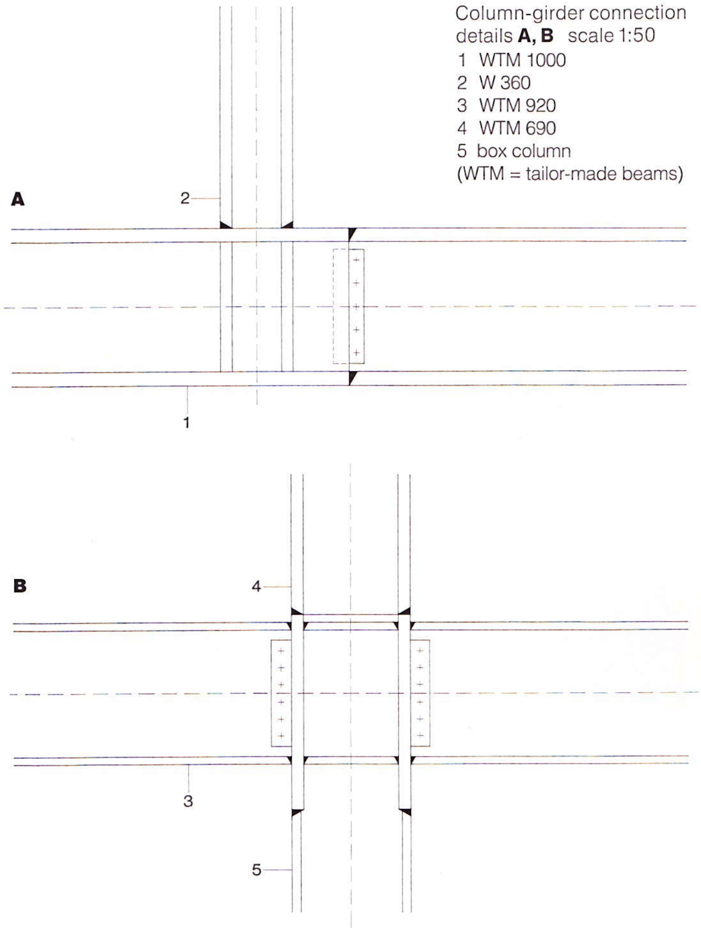
The 39-storey office tower with its dramatic curve and highly articulated facade in Venetian Gold and Verde Fountain granite, with saphire tinted glass creates a unique profile on the Los Angeles skyline while providing over 72.000 rentable square metres of efficient interior office space and spectacular views.
One of the main features of the building is the 26 m wide elliptic curved facade at the south-east corner. This facade spans uninterrupted with columns at the first storey. The exterior columns at ground level form an arcade with an unbraced height of 12,20 m. There is a setback at the 36th storey. The floor plate is column-free between the core and the exterior glass line. However, due to the L-shaped core only one column was allowed to be used at the south-east quarter of the building to carry the gravity loads. This column stops above the lobby area to accommodate the main corner entrance. The column gravity loads are delivered to a 14,00 m long, 3,25 m deep, tapered plate girder above the entrance ceiling.

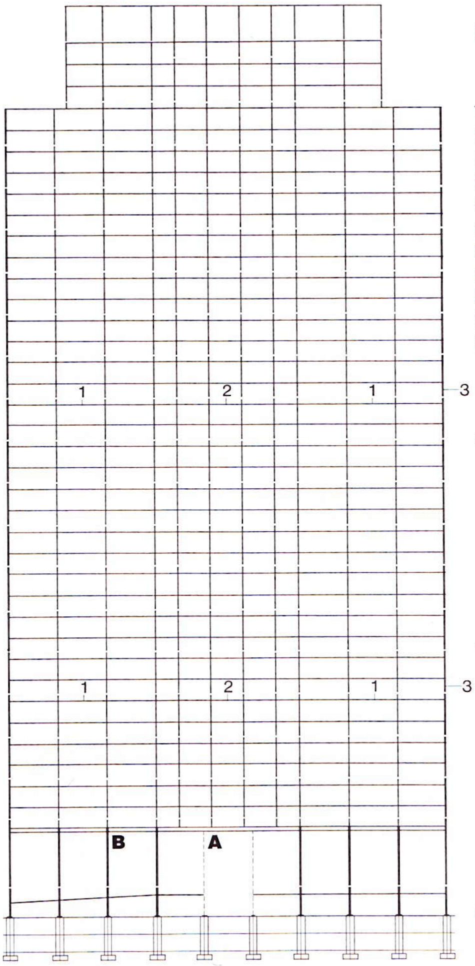
A dual framing system

The floor framing of the building consists of partial composite steel beams 3,00 m on centre with 51 mm metal deck and 83 mm lightweight concrete fill. Below grade framing system is reinforced concrete beams and slabs. Located in a highly seismic area, the tower had to be designed to withstand the maximum ground shaking predicted in the site, as well as provide for wind induced accelerations, which may adversely affect occupant comfort. Accordingly, a structural steel dual system was selected for final design. This system consists of a perimeter tube integrated with six interior core eccentric bracing frames. The perimeter tube consists of a ductile steel moment frame up to the 36th storey. Above this level the tube is divided into separate frames.

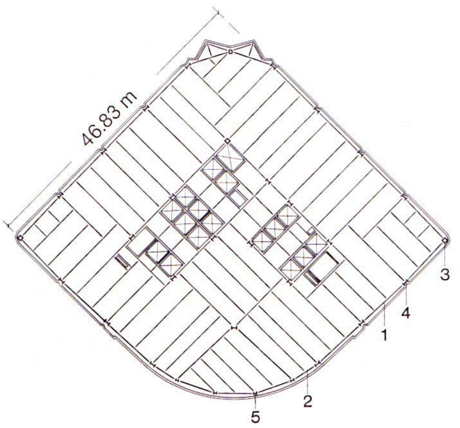
1999 Avenue of the Stars, Los Angeles

Column-girder connection details **A, B** scale 1:50
1 WTM 1000
2 W 360
3 WTM 920
4 WTM 690
5 box column
(WTM = tailor-made beams)





The selected system has an advantage from both stiffness and ductility aspects. In addition to the ductile moment frames, the shear links of the eccentric bracing frames systems act as fuses for seismic energy dissipation.



Structural frame elevation of perimeter tube and framing plan 7th to 13th floor
1 W 920
2 WTM 920
3 box column
4 WTM 690
5 W 360
(WTM = tailor-made beams)

Architects:
Lee Harris Pomeroy Associates/
Abramovitz Kingsland Schiff,
Manhattan, N.Y.

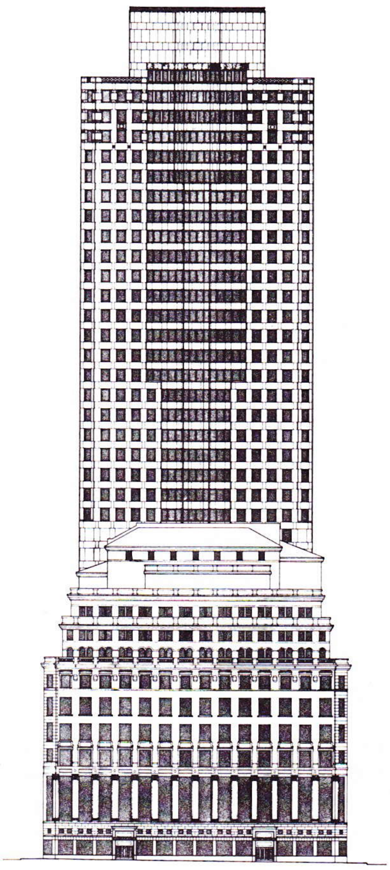
Structural Engineer:
The Office of Irwin G. Cantor P.C.,
Manhattan, N.Y.

Developer:
The Swiss Bank Corporation
and Saks & Company,
Manhattan, N.Y.

Two-buildings-in-one

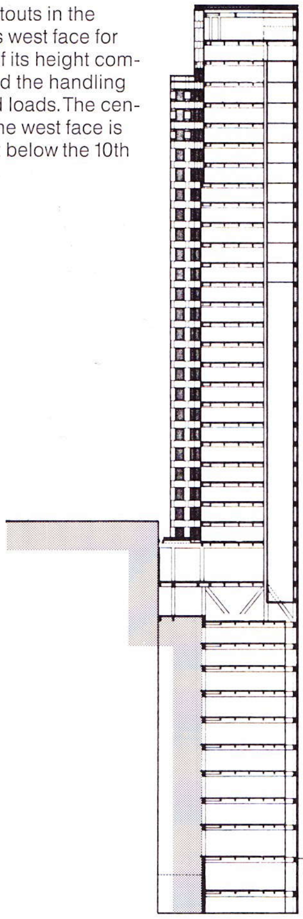
The Swiss Bank tower was placed to the rear of the historic department store, and designed to respect the classical architecture of the Saks' building while allowing the tower to rise above the new 9-storey store extension to Saks' Fifth Avenue on axis with Rockefeller Center. Extraordinary construction efforts were necessary to satisfy two owners with radically different requirements. The different column locations in the two-buildings-in-one required an innovative column-transfer system. While Saks wanted exceptionally long 18,60 m spans on its retail floors, in the Swiss Bank above, in contrast, in the same east-west orientation of the Saks' spans, the spans are only 14,00 m.

West elevation

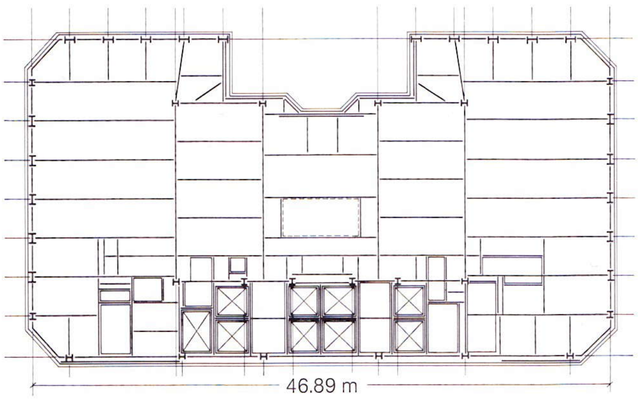


Swiss Bank – Saks, New York

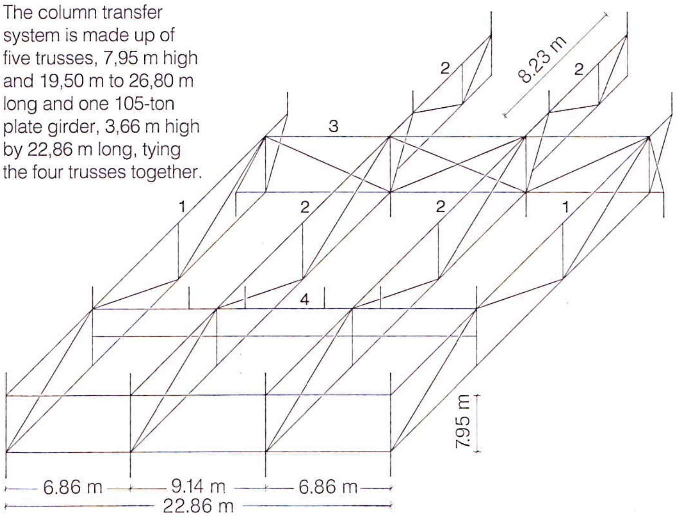
The cutouts in the tower's west face for most of its height complicated the handling of wind loads. The centre of the west face is cut out below the 10th storey.



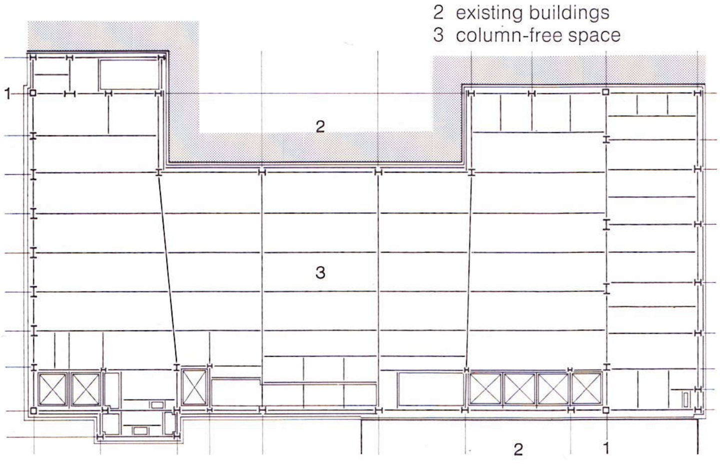
Framing plan: 15th to 20th floor



The column transfer system is made up of five trusses, 7,95 m high and 19,50 m to 26,80 m long and one 105-ton plate girder, 3,66 m high by 22,86 m long, tying the four trusses together.



Framing plan: 4th to 7th floor
1 corner column
2 existing buildings
3 column-free space



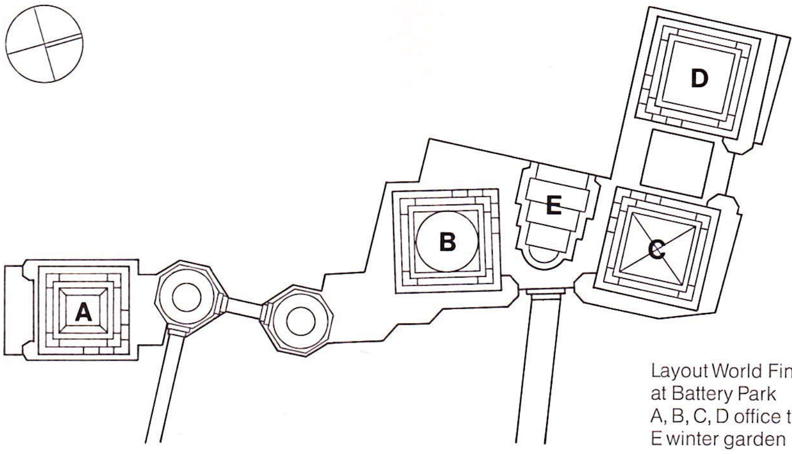
Structural variety

The transfer system covers the central portion of the upper tower and supports the elevators and on the opposite side of the building, a 3-storey high portion of the Swiss Bank that cantilevers 8,23 m over Saks' escalator tower. The space within the 2-storey high truss grid houses the building's mechanical equipment. In the upper, Swiss Bank portion, wind loads are resisted by a hybrid structural frame, consisting of exterior tube frames on three of the tower's four sides, and an exterior braced frame on the fourth side. The centre of the west face is cut out below the 10th storey. From the 13th storey up to the top is another cutout. Because of these cutouts the building has a hybrid wind frame below the 14th storey.

Architects:
Cesar Pelli & Associates,
New Haven, Connecticut

Structural Engineer:
Thornton-Tomasetti, P.C.
New York, N.Y.

Developer:
Olympia & York
Equity Corporation,
Toronto, Canada



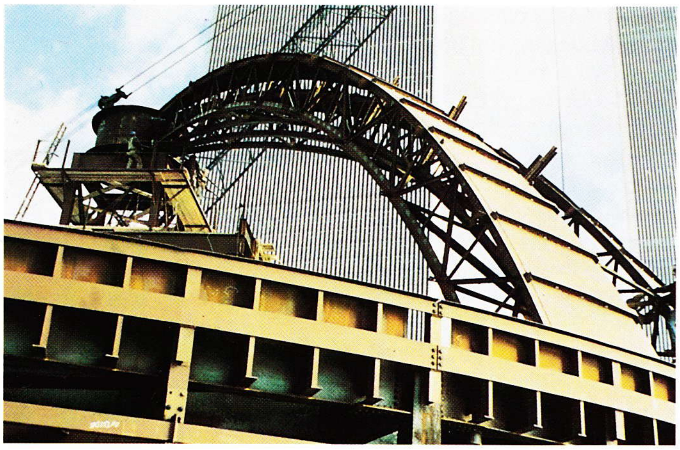
Layout World Financial Center at Battery Park
A, B, C, D office towers
E winter garden

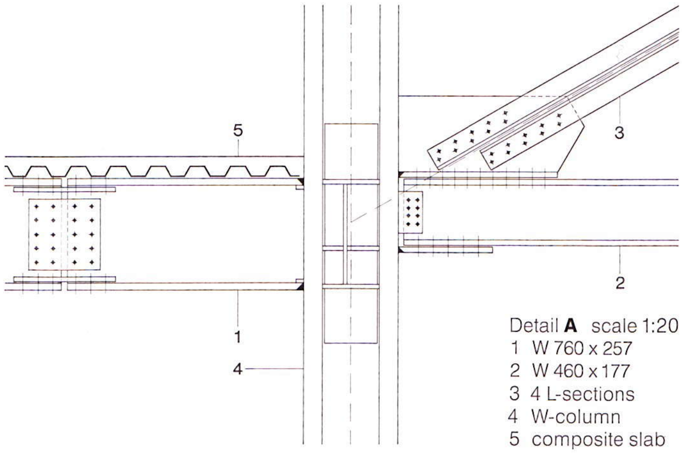
Architectural versatility

The World Financial Center at Battery Park City is a group of architecturally coordinated skyscrapers, landscaped plazas and winter garden. The four reflective glass and granite office towers rise from a continuous granite-sheathed base and make a transition towards entirely reflective glass.
The tops of the towers are designed as sculptured forms–spheres, pyramids and prisms. What has become to be called the Building B is unique in that it is the only one to feature a 45,72 m diameter copper dome supported by a series of steel trusses. Building B has a ten-storey podium stretching from the tower to the nine-storey octagonal structure of the gatehouse located to the south and forming the main entry at Liberty Street.

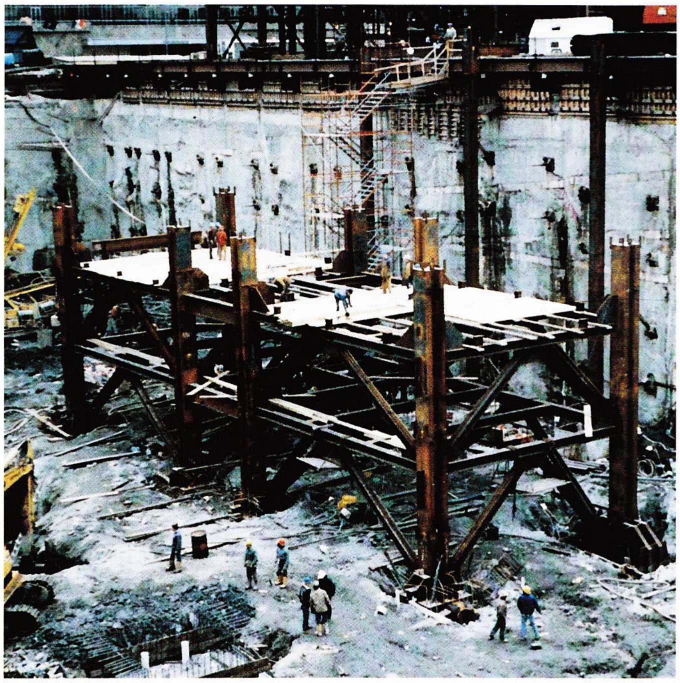
A case for steel

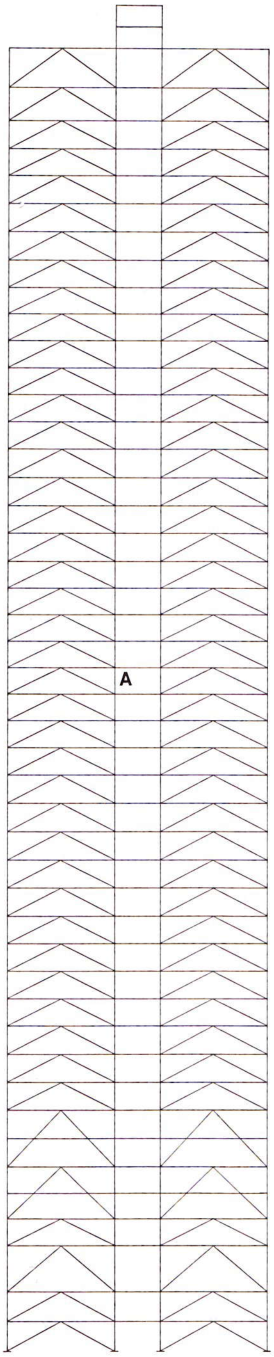
Structural steel was selected for Building B because of several programme requirements, including electrified floor system flexibility to accommodate tenant requirements and a transfer girder that had to be accommodated in the 91 cm ceiling sandwich. The 45-storey tower has a 4,57 m setback at the 26th storey. Instead of carrying columns all the way down and having a column spacing of 4,57 m, the upper perimeter columns were picked up on transfer girders allowing a 9,14 m by 9,14 m grid to be employed below the 26th storey.
The dome framing consists of 48 arch-trusses spanning between a tension ring and a compression hub. The arch-trusses are spaced approximately 2,74 m on centre at the tension ring and converge at the compression hub. Eight rows of concentric bridging provide bracing for the archtruss bottoms chords. Simple field connection details connect the arch-truss to the tension ring and the compression hub.





Detail **A** scale 1:20
1 W 760 x 257
2 W 460 x 177
3 4 L-sections
4 W-column
5 composite slab





Structural elevation of the two connected braced cores, line **B**

40th floor

30th floor

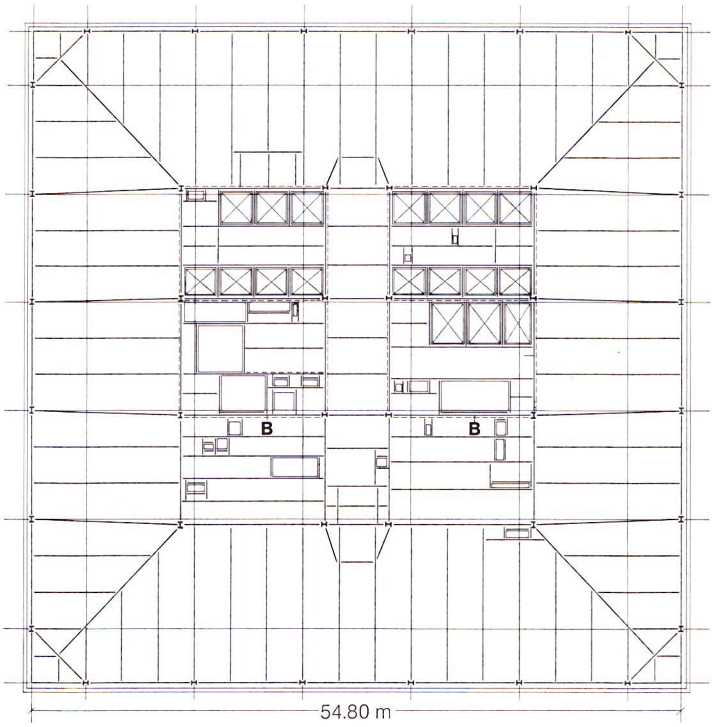
20th floor

10th floor

street level

All lateral forces are resisted by the two connected cores acting as giant vertical cantilever. The wind bracing consists of K-type bracing arranged and designed to minimize the loss of rentable area.

Framing plan



The general framing of the podium consists of structural steel beams spaced 3,05 m on centre framing into girders and spanning 9,14 m. The same framing system is used for the 20th to 26th floor. The framing above the 26th floor consists of floor beams spaced 3,05 m on centre and spanning approximately 12,80 m between the spandrel beam and the centre core. The beams are designed to act composite with the concrete slab of the floor construction.

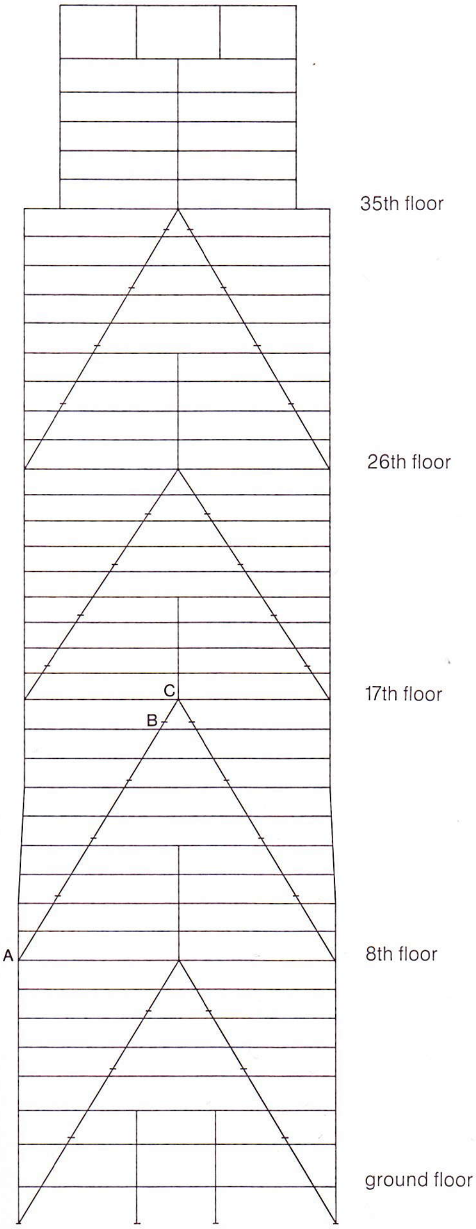
Architects:
Kohn Pedersen Fox Associates,
Manhattan, N.Y.

Structural Engnieer:
Severud Associates,
Manhattan, N.Y.

Developer:
Shearson Lehman Hutton,
Manhattan, N.Y.

Extraordinary dimensions

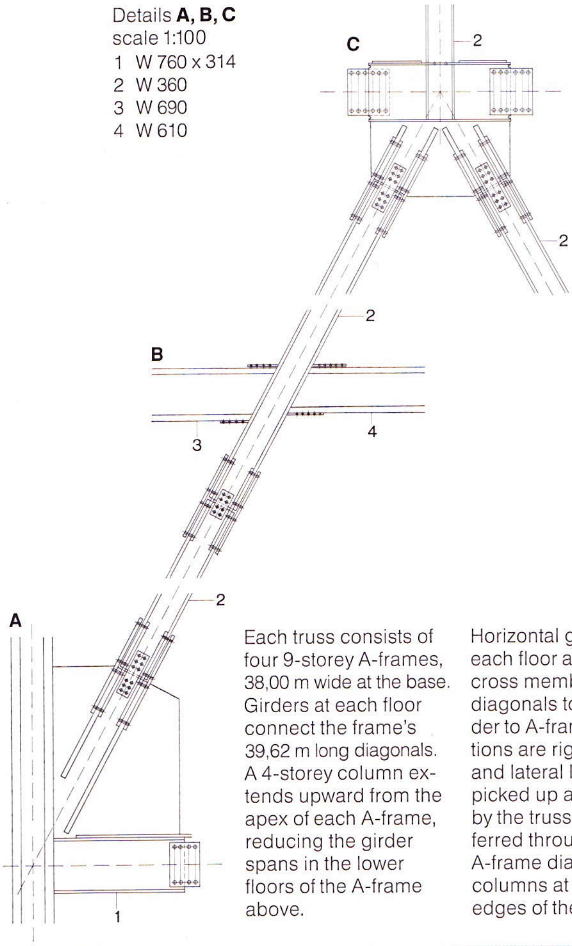
The big building, not only in height, but in plan, measuring 81 x 50 m is located on Manhattan's Hudson River shore near the World Trade Center. There are entrances on the east, west and north sides, and the three lobbies are interconnected. These lobbies and interconnections have about 213 running metres marble wall, necessitating an extraordinary extensive substructure of miscellaneous iron framing to support the marble. Thanks to the use of steel, accommodating changes late in the design was far easier, faster and at lower cost than would have been possible with concrete. Some of the major changes were the addition of one complete floor at the mid-height of the building, an auditorium at the 26th floor with transfer girders above the clear span space, a reinforced floor for an aerobic area, new openings for connecting stairs, framed pits at the kitchen, and reframing



Wind bracing north-south: a-bracing over 9 floors with columns on the apex to reduce the span of the horizontal girder. Vertical and lateral loads are picked up at each floor by the truss and transferred through the A-frame diagonals to megacolumns at the outer edges of the "A".

Shearson Lehman Hutton Plaza, New York

Details **A, B, C**
scale 1:100
1 W 760 x 314
2 W 360
3 W 690
4 W 610



supports for the exterior precast walls. The general-use office is believed to be the largest in Manhattan with raised floors throughout for computer cabling and wiring.

The superiority of steel

Although the building has characteristics that would have made concrete less costly than usual, for example, relatively close column spacing, rectangular floor plans, central rectangular core, moderate floor-to-floor heights and raised floors for electrification, a steel frame was fully justified for its superior advantages. The problem of the heavy loads of the high speed double decked elevators was solved by including elevator support beams at two intermediate floors as well as at the roof. Further, the requirement for column-transfer beams for the setbacks at the upper floors were best answered with steel. Both a braced core and moment frames are used to resist lateral wind load. The vertical trusses running full width and height of the building were placed 4,57 m from the end walls, so as not to diminish the magnificent view. Moreover, at this location the diagonal and vertical members of the truss could be hidden by office partitions. To limit the number of vertical and diagonal members, the truss was designed as a simple "A" frame. The wind load in the long direction of the building is resisted by conventional crossbracing in the cores plus moment frames.

Each truss consists of four 9-storey A-frames, 38,00 m wide at the base. Girders at each floor connect the frame's 39,62 m long diagonals. A 4-storey column extends upward from the apex of each A-frame, reducing the girder spans in the lower floors of the A-frame above.

Horizontal girders at each floor are used as cross members to tie diagonals together. Girder to A-frame connections are rigid. Vertical and lateral loads are picked up at each floor by the truss and transferred through the A-frame diagonals to columns at the outer edges of the "A".

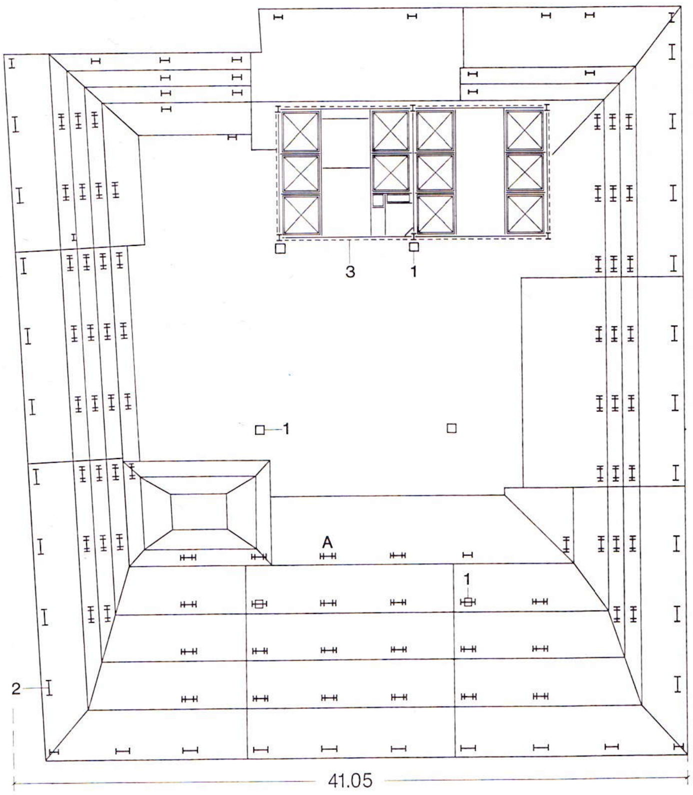
Framing plan 19th to 22nd floor
1 wind bracing north-south
2 bracing core

Architects:
Kevin Roche John Dinkeloo
& Associates, Hamden, Connecticut

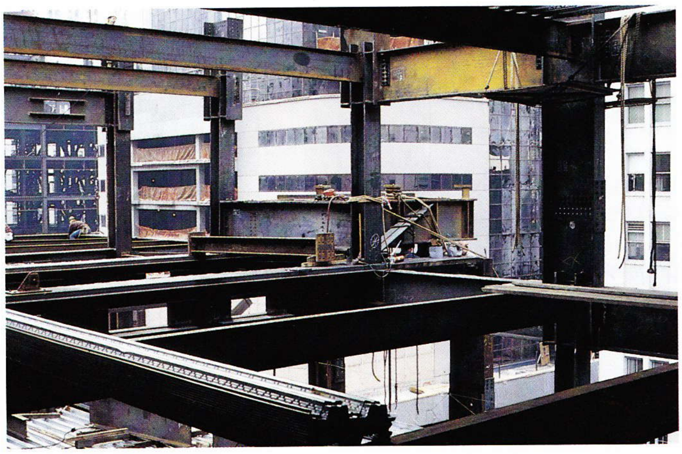
Structural Engnieer:
Weiskopf & Pickworth,
Consulting Engineers, Manhattan, N.Y.

Developer:
Solomon Equities, Inc.,
New York, N.Y.

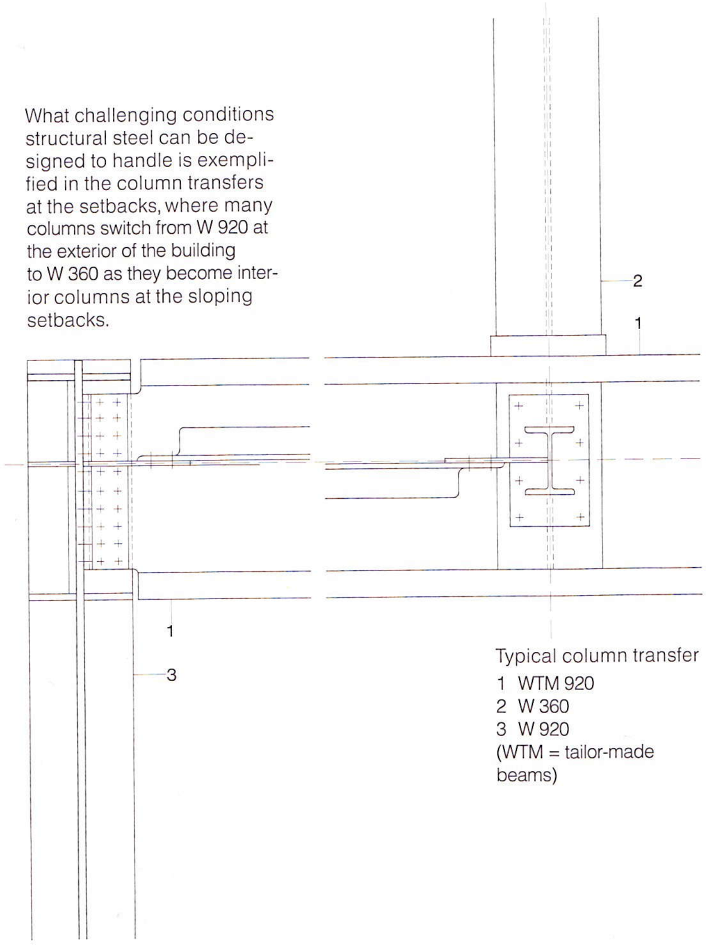
750 Seventh Avenue, New York



Column layout including core
1 six interior non-core columns on lower levels
2 W 920 section
3 braced core



What challenging conditions structural steel can be designed to handle is exemplified in the column transfers at the setbacks, where many columns switch from W 920 at the exterior of the building to W 360 as they become interior columns at the sloping setbacks.



Typical column transfer
1 WTM 920
2 W 360
3 W 920
(WTM = tailor-made beams)

A tapering spiral shape

This 35-story tower features setbacks that spiral around the building and culminate at the top with a 36,58 m high spire. The only way to satisfy the architectural design and the owner's requirements for column-free space was to use transfer girders. While three or four or even six or seven transfer girders are not uncommon, the tower has 84 column transfers on 26 perimeter columns. The use of column transfers led to complicated column connections, whose complexity is increased by the fact that many columns switch from W 920 at the exterior of the building to W 360 as they become interior columns at the sloping setbacks. At a single location, some columns had to be designed for a change in section, a spandrel moment connection, a moment connection to the girder that frames at right angles to the spandrel, and horizontal X-bracing. Where ductwork had to pass through the girders, the many penetrations make the girders look like "Swiss-cheese". Especially near the building core, these conditions meant girder penetrations as large as 1,82 m wide and 406 mm high through a 1,067 m deep girder.

A challenging structural design

From outside the building, large vision panels are separated by opaque grey glass bands that closely shadow the location and dimensions of the structure's exterior tube frame. The horizontal opaque bands are 0,91 m deep, the depth of the floorceiling sandwich; the vertical opaque bands are 1,52 m wide, little more than the width of the 914 mm columns that are spaced on 4,57 m centres along the building perimeter plus the column covers. This building is one of the first in New York City where the glass in the curtainwall is attached to its supporting members solely by adhesive structural silicone.

Design Architect:
I. M. Pei & Partners, New York, N.Y.

Executive Architect:
Welton Becket Associates, Los Angeles

Structural Engineer:
CBM Engineers, Inc., Houston

Developer:
Maguire Thomas Partners
and Pacific Enterprises, Los Angeles

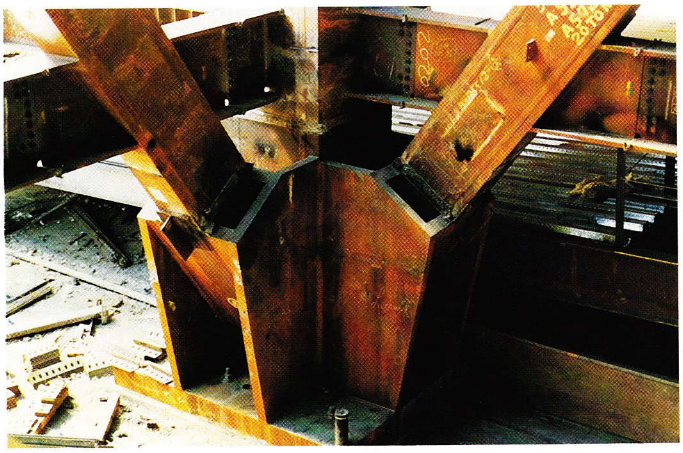
The tallest tower in LA

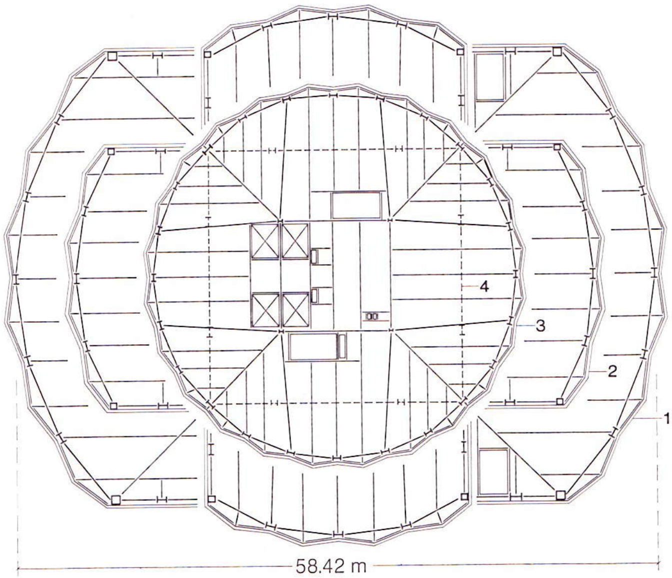
Roughly circular in plan at the base, the building changes shape five times as it steps back at the 47th, 56th, 60th, 69th and 73th floor. A three level difference in elevations between streets on north and south sides is made up with beautifully landscaped Spanish style steps, which are seismically isolated from the main tower structure, and fountains. Called a signature building for the city of Los Angeles, the granite clad building with its faceted exterior will contain about 1.4 million square feet of office space.

Conflicting structural demands

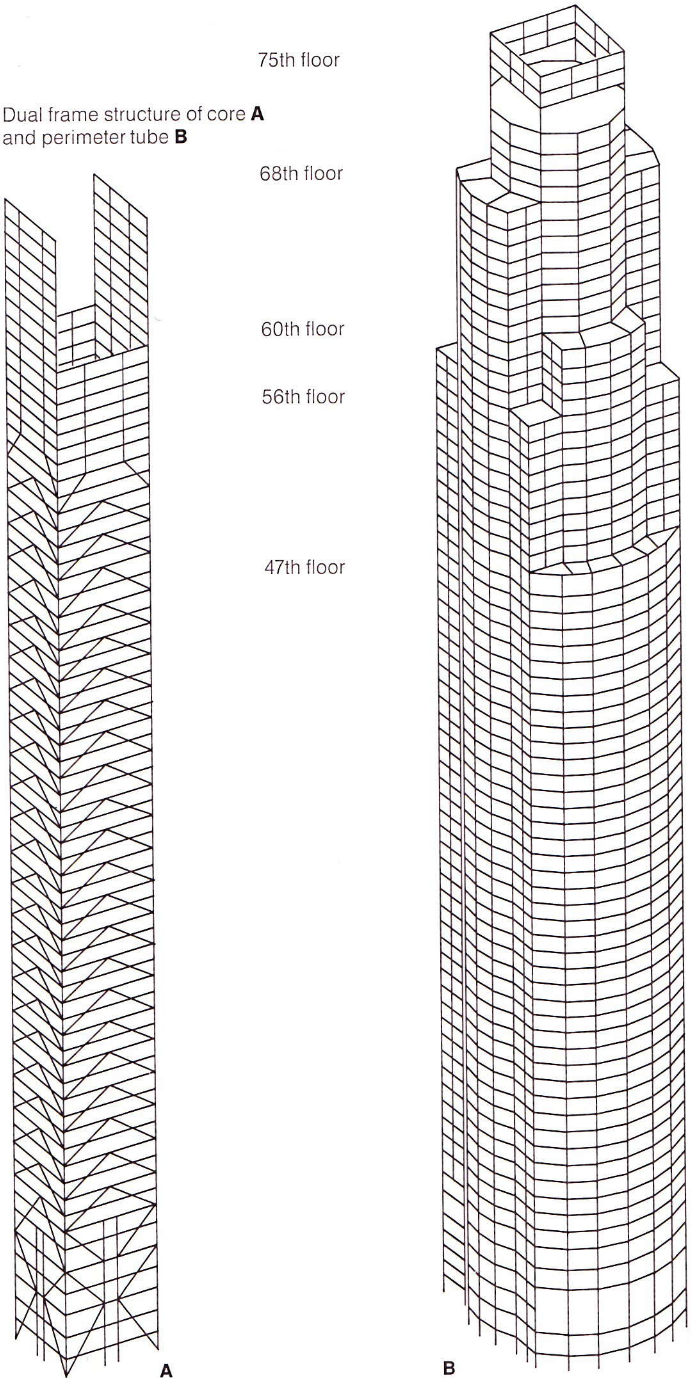
To arrive at the most economical design for the steel structure, due consideration had to be given to both seismic and wind-loading factors. The conflicting demands of optimal seismic vs optimal high-rise design resulted in a structure that is really two structures interacting with each other, with enough ductility to accommodate the randomly variable jerks of an earthquake yet stiff enough to ensure occupant comfort during high winds. The dual system that was selected consists of steel columns ringing the building's perimeter creating a ductile, moment-resisting tube. It is the 6,80 m^2 core, formed by four massive 1,20 x 1,20 m steel core columns connected by chevron bracing that provides stiffness up to the 54th floor. At the 54th floor, the chevron bracing drops out of the core. At this point, there are no columns between the core and the perimeter tube, and the perimeter columns act as the main lateral bracing mechanism. The core structure is supported on a concrete mat foundation. The perimeter frame columns are supported on circumferential strip footings that tie into the core mat. Curtain walls were designed to sustain practically no damage during earthquakes. The trusses that support the cladding were designed to move with the maximum interstorey drift.

First Interstate World Center, Los Angeles





Floor framing plans (selection)
1 perimeter of 8th to 46th floor
2 perimeter of 57th to 59th floor
3 floor framing plan 70th to 71st floor
4 braced core lower levels



Dual frame structure of core **A** and perimeter tube **B**

The perimeter structure is a ductile, moment resisting frame. The interior core is 22,55 x 22,55 m wide and 68 storeys high. Box colums at each corner of this core have design gravity loads of 10.000 t. Lateral support is provided by a series of 2-storey chevron braces, spanning each of the four sides of the core. The chevron braces are being used in a high seismic region for the first time. At the 54th floor the chevron braces are replaced by free-spanning Vierendeel girders. Between the core frame and the perimeter frame, concrete floor slabs on steel beams span up to 16,76 m.

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