Special session 4: High rise buildings

Objekttyp: **Group**

Zeitschrift: **IABSE congress report = Rapport du congrès AIPC = IVBH Kongressbericht**

Band (Jahr): **14 (1992)**

PDF erstellt am: **12.07.2024**

Nutzungsbedingungen

Die ETH-Bibliothek ist Anbieterin der digitalisierten Zeitschriften. Sie besitzt keine Urheberrechte an den Inhalten der Zeitschriften. Die Rechte liegen in der Regel bei den Herausgebern.
Die auf der Plattform e-periodica veröffentlichten Dokumente stehen für nicht-kommerzielle Zwecke in Lehre und Forschung sowie für die private Nutzung frei zur Verfügung. Einzelne Dateien oder Ausdrucke aus diesem Angebot können zusammen mit diesen Nutzungsbedingungen und den korrekten Herkunftsbezeichnungen weitergegeben werden.
Das Veröffentlichen von Bildern in Print- und Online-Publikationen ist nur mit vorheriger Genehmigung der Rechteinhaber erlaubt. Die systematische Speicherung von Teilen des elektronischen Angebots auf anderen Servern bedarf ebenfalls des schriftlichen Einverständnisses der Rechteinhaber.

Haftungsausschluss

Alle Angaben erfolgen ohne Gewähr für Vollständigkeit oder Richtigkeit. Es wird keine Haftung übernommen für Schäden durch die Verwendung von Informationen aus diesem Online-Angebot oder durch das Fehlen von Informationen. Dies gilt auch für Inhalte Dritter, die über dieses Angebot zugänglich sind.

Ein Dienst der *ETH-Bibliothek*
ETH Zürich, Rämistrasse 101, 8092 Zürich, Schweiz, www.library.ethz.ch

http://www.e-periodica.ch

Special Session 4

High Rise Buildings

Maisons hautes

Hochhäuser

Organizer: Walter P. Moore,
USA

Leere Seite
Blank page
Page vide

The Miglin-Beitler Tower Chicago, IL (USA)

La tour Miglin-Beitler de Chicago, IL (USA)

Der Miglin-Beitler-Turm in Chicago, IL(USA)

Charles H. THORNTON
Chairman and Principal
Thornton-Tomasetti
Engineers
New York, NY, USA

Dr. Thornton is a recognised expert in many structural areas such as long-span structures, brittle fracture, lamellar tearing, creep and shrinkage of concrete and seismic and dynamic analyses. He received a bachelor's degree in Civil Engineering from Manhattan College, a master's degree in Civil Engineering and a doctorate in Philosophy in Structural and Engineering Mechanics from New York University.

Jagdish PRASAD
Senior Vice-Pres. and Princ.
Thornton-Tomasetti
Engineers
New York, NY, USA

Mr. Prasad is in charge of the production of all design jobs in the firm and has been project manager on numerous projects such as Bally's Park Place Casino Hotel, Vanderbilt University Medical Center and Fifth Avenue Place in Pittsburgh. He received a bachelor's degree in Civil Engineering from Howard University and a master's degree in Civil Engineering from Lehigh University.

Udom HUNGSPRUKE
Vice-Pres.
Thornton-Tomasetti
Engineers
New York, NY, USA

Mr. Hungspruke has been project manager for such projects as the Northwestern Atrium Center, United Terminal Complex at O'Hare Airport, and One Liberty Place in Philadelphia, Pa. Mr. Hungspruke received a Bachelor of Science degree in Civil Engineering from Mapua Institute of Technology.

Robert P. DeSCENZA
Associate
Thornton-Tomasetti
Engineers
New York, NY, USA

As Associate, Mr. DeScenza has been responsible for the analysis and structural design of several large projects, including the Northwestern Atrium Center and the United Terminal Complex at O'Hare Airport. Mr. DeScenza holds a Bachelor's degree in Civil Engineering from The Cooper Union.

SUMMARY
A cruciform tube structure provides a safe, elegant, efficient and constructable solution to the challenge of designing the world's tallest building, the Miglin-Beitler Tower in Chicago, Illinois. The proposed structural solution combines the erection speed of concrete construction, the flexibility for future change and the efficiency for horizontal spans of a steel floor system, and the superior dynamic acceleration response of a composite lateral load resisting structural system.

RÉSUMÉ
Une structure tubulaire cruciforme fournit une solution sûre, élégante, efficiente et réalisable pour la conception de l'immeuble le plus haut du monde, à savoir la tour Miglin-Beitler de Chicago, dans l'Illinois. Le modèle structural proposé combine la vitesse de mise en œuvre de la construction en béton, la flexibilité pour des modifications futures, l'efficience des portées horizontales du système de plancher en acier, ainsi que la réponse maximale d'accélération dynamique donnée par un système structural résistant aux forces latérales mixtes.

ZUSAMMENFASSUNG
Für das höchste Gebäude der Welt, den Miglin-Beitler-Turm in Chicago, wurde eine Röhrenstruktur in Kreuzform gewählt, die für dieses ambitiöse Bauwerk eine sichere, elegante, effiziente und ausführungsfreundliche Lösung bietet. Der vorgeschlagene Entwurf kombiniert den schnellen Baufortschritt des Betonbaus mit der Freiheit grosser Spannweiten und späterer Umbaumöglichkeit eines Stahlgeschosssystems. Dieses hybride Aussteifungssystem gegenüber Horizontalkräften gewährleistet ein überlegenes, dynamisches Verhalten.

1. INTRODUCTION

The city of Chicago, Illinois will gain the honor and distinction as the holder of the worlds two tallest buildings with the construction of the Miglin-Beitler Tower. At 610 metres (1,999 feet 11 1/2 inches) to the tip of its spire, the Miglin-Beitler Tower will provide a regal landmark for the Chicago skyline and establish new records as the world's tallest building and the world's tallest non-guyed structure. Its 176,500 square metres of total floor area are to be split up into small plates that will enable smaller sized firms to rent entire floors (see figure 1). At the present time, the owner has applied for foundation permits that will allow a start of construction of the building in mid 1992.

The tower is to have 12 levels of above grade parking with commercial office floors above. At the top of the tower will be a multistory observation deck. From the upper observation level at 462 metres, observers will see the 443 metre Sears Tower below. The steel framed spire will surpass the 555 metre CN Tower in Toronto.

A simple and elegant integration of building form and function has emerged from close cooperation of architectural, structural and development team members. The resulting cruciform tube scheme offers structural efficiency, superior dynamic behavior, ease of construction and minimal intrusion at leased office floors for this 125 story office building.

2. STRUCTURAL SYSTEM

The structural system for the Miglin-Beitler Tower is a composite system that exploits the advantages of both steel and concrete to solve the challenges of a 610 metre tall building. The challenge put to the design team was to come up with an economical and buildable structural frame capable of resisting vertical and lateral loads for this supertall building. The Chicago area is subject to wind forces based upon a 120 kilometre per hour basic wind speed and seismic forces based upon seismic Zone 1. The challenge was met by taking advantage of the mass and stiffness of the high strength concrete that is available in the Chicago area and combining it with the advantages of a structural steel floor system with its inherent strength, speed of construction and flexibility to allow tenant changes (see figure 2). The building has a density in the range of 2.5 to 3 kN/m^3. 68,000 $Metre^3$ of concrete, reinforced with 9,100 metric tons of steel, will be used to build the Miglin-Beitler Tower. The steel spire will require approximately 9,000 metric tons of rolled structural steel framing.

The cruciform tube structural system consists of six major components, as listed below:

1. A 19 metre by 19 metre concrete core has walls of varying thickness. The interior cross walls of the core are generally not penetrated with openings. This contributes significantly to the lateral stiffness.

2. Eight cast-in-place concrete fin columns are located on the faces of the building and extend up to 6 metre beyond the 43 x 43 metre tower footprint. They vary in dimension from 2 metre by 11 metre at the base to 1.7 metre by 5 metre at the middle and 1.4 metre by 4.5 metre near the top.

3. Eight link beams connect the four corners of the core to the eight fin columns at every floor. These reinforced concrete beams are haunched at both ends for increased stiffness

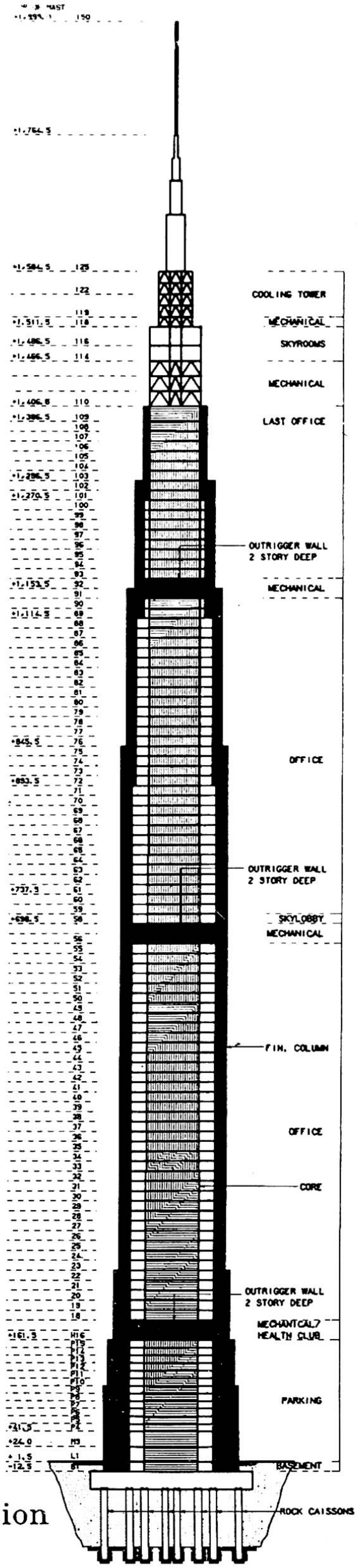


FIGURE 1

Building Elevation

and reduced in depth at mid-span to allow for passage of mechanical ducts. By linking the fin columns and core they enable the full width of the building to act in resisting lateral forces. In addition to link beams at each floor, sets of two-story-deep outrigger walls are located at levels 17, 57 and 89. These outrigger walls enhance the interaction between exterior fin columns and the core.

4. The conventional structural steel composite floor system has 457 mm deep rolled steel beams spaced at approximately 3 metre on center. A slab of 8 cm deep 20 gage corrugated metal deck and 9 cm of stone concrete topping spans between the beams. The steel floor system is supported by the cast-in-place concrete elements.

5. Exterior steel vierendeel trusses frame the entire perimeter of the building. At each of the four faces between fin columns the vierendeels consist of the horizontal spandrels and two vertical columns. At the corners, steel vierendeel trusses are used to pick up each of the four cantilevered corners of the building.The vierendeel trusses provide additional resistance to lateral forces as well as improving the resistance of the entire structural system to torsion. In addition, the trusses transfer dead load to the fin columns to eliminate tensile and uplift forces in the fin columns. All corner columns are eliminated providing for corner offices with undisturbed views. Connections between the steel vierendeel trusses and the concrete fin columns are typically simple shear connections which minimize costs and expedite erection.

6. A 137 metre tall steel framed tower tops the building. This braced frame is to house observation levels, window washing, mechanical equipment rooms and an assortment of broadcasting equipment.

Concrete strengths for the concrete core and fin columns vary from 69 MPa to 96.5 MPa. Because of the mass required to minimize overturning and perception of motion, stresses in concrete are low. High concrete strength is required, however, to give high modulus of elasticity (up to 4.83×10^7 kPa) for increased stiffness (see figure 3).

3. LATERAL FORCES

The proposed building and structural system have undergone extensive wind tunnel testing at RWDI in Guelph, Ontario. Pressure tap models, pedestrian level studies, high frequency force balance and aeroelastic models have been used to determine the static and dynamic behavior of the project under wind loadings. The high frequency force balance results were in very close agreement to the results from the aeroelastic model.

Working in parallel with Thornton-Tomasetti's three dimensional static and dynamic computer analyses, RWDI has accounted for the structural properties, mass and damping of the building in their wind studies and confirmed that the proposed structural system provides ample resistance to all expected wind loads.

The design has also received a superior performance rating in its ability to virtually eliminate occupant perception of wind movements and accelerations. Results from the force balance model indicate that the upper floors of the building will experience accelerations in the range of 26 millig's, while the more refined aeroelastic model indicates accelerations below 23 millig's. Both results fall within acceptable ISO acceleration criteria.

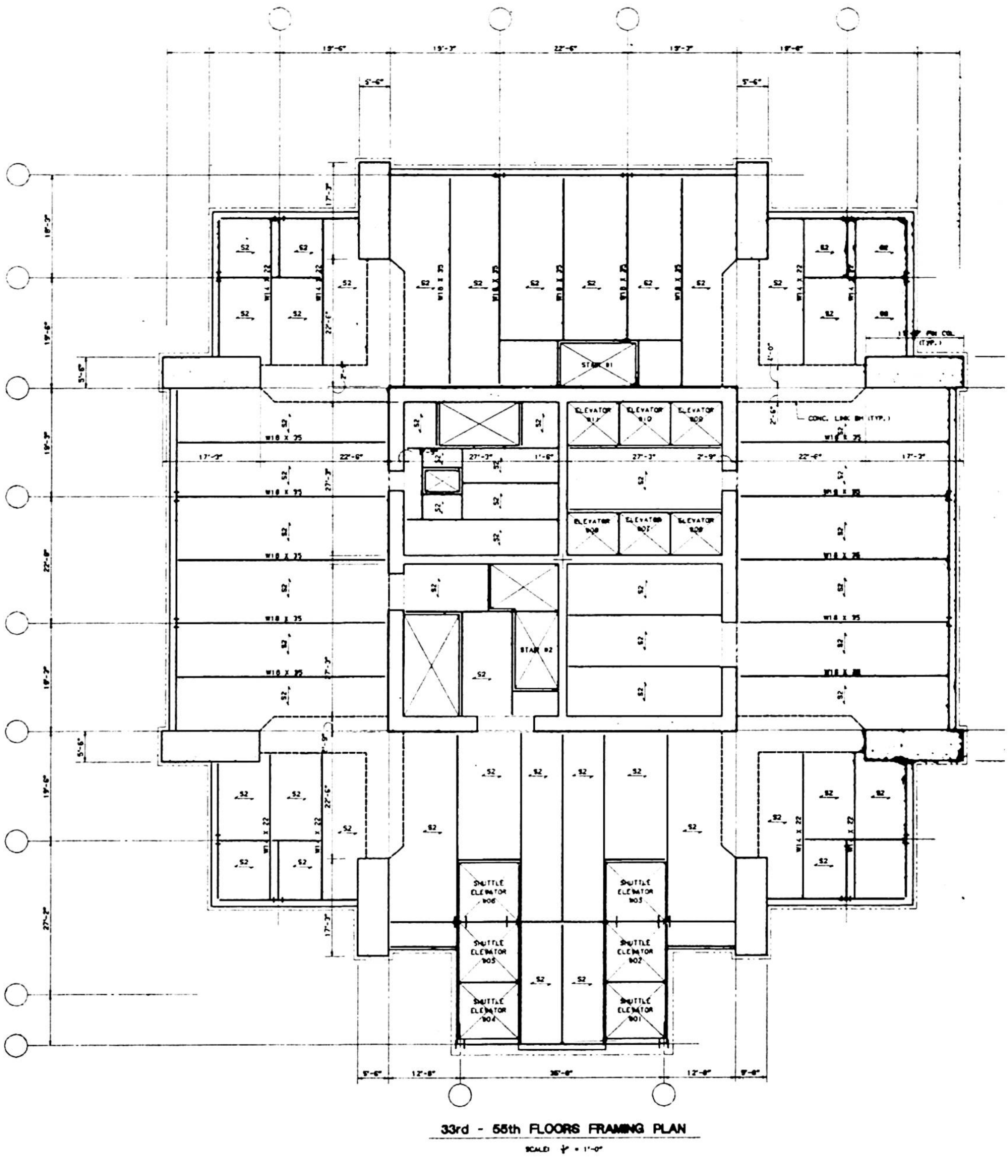
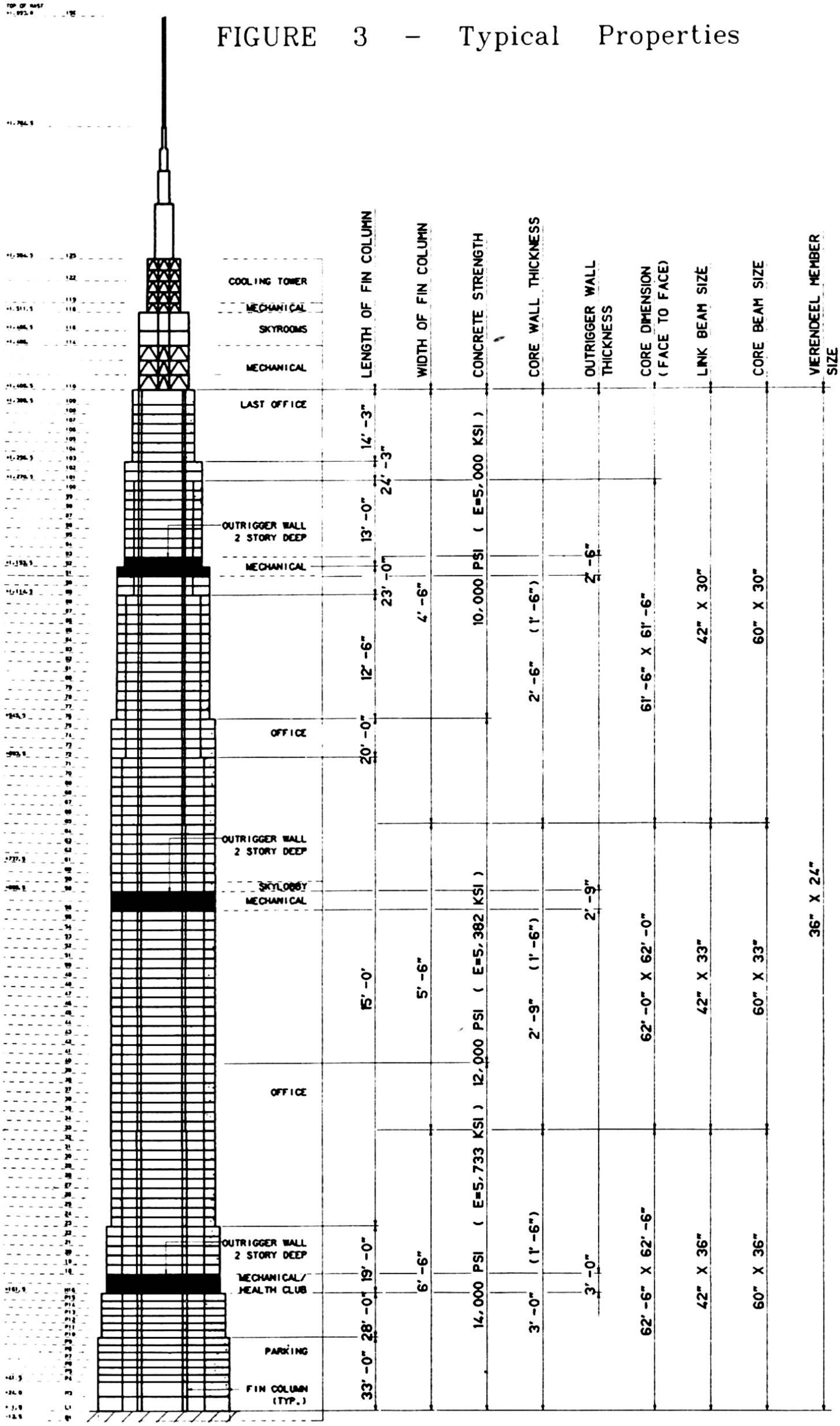
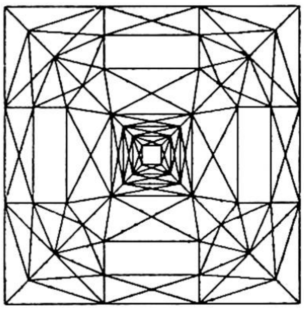


FIGURE 2 – Typical Floor Plan

FIGURE 3 - Typical Properties





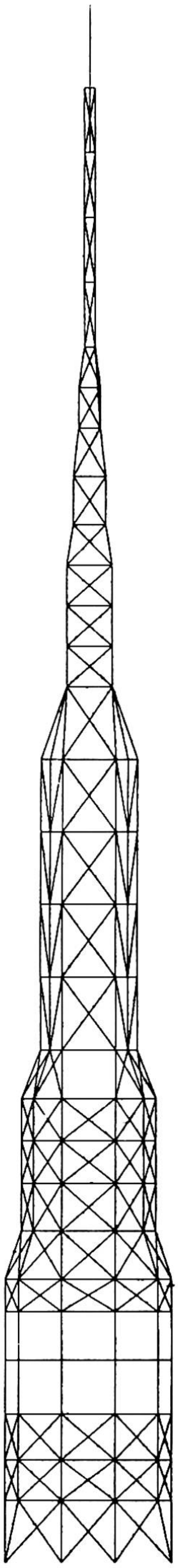


FIGURE 4 – Spire Model

Sixteen independent two-dimensional and three-dimensional static and dynamic computer analyses have been used by Thornton-Tomasetti in determining the viability and acceptability of the proposed structural system. Three separate computer programs, EASE-II, SAP90 and ETABS, have provided parallel checks of the accuracy and adequacy of the computer simulations. The EASE-II models were run on a super minicomputer while the ETABS and SAP90 analyses were run on 486-based microcomputers.

The parallel sets of models were compared to validate computer approaches. Static displacements and dynamic mode shapes from the two sets of analyses were in very close agreement. Overall displacements, modal shapes and natural frequencies differed by less than ten percent. The primary periods for the first lateral modes are in the range of 9 seconds. The first torsional mode has a period around 2.4 seconds. Displacements at the uppermost office floor of the building are in the range of 630 mm, which result in a drift index of 1/650.

Although only UBC Zone 1 is applicable, the structural system was investigated for the effects of a UBC Zone 2 earthquake and has been found satisfactory. This is not surprising because seismic building response drops quickly for long-period structures while wind effects increase with increased height.

4. FOUNDATIONS

The foundation system proposed for this project uses caissons varying from 2.6 metre to 3.2 metre in diameter. Each 29 metre long caisson has a straight shaft and a rock socket a minimum of 2 metre into competent rock. Caisson concrete is to have an ultimate compressive strength of 93 MPa to take into advantage the quality rock that was encountered at the site, with an allowable bearing pressure of 23 MPa.

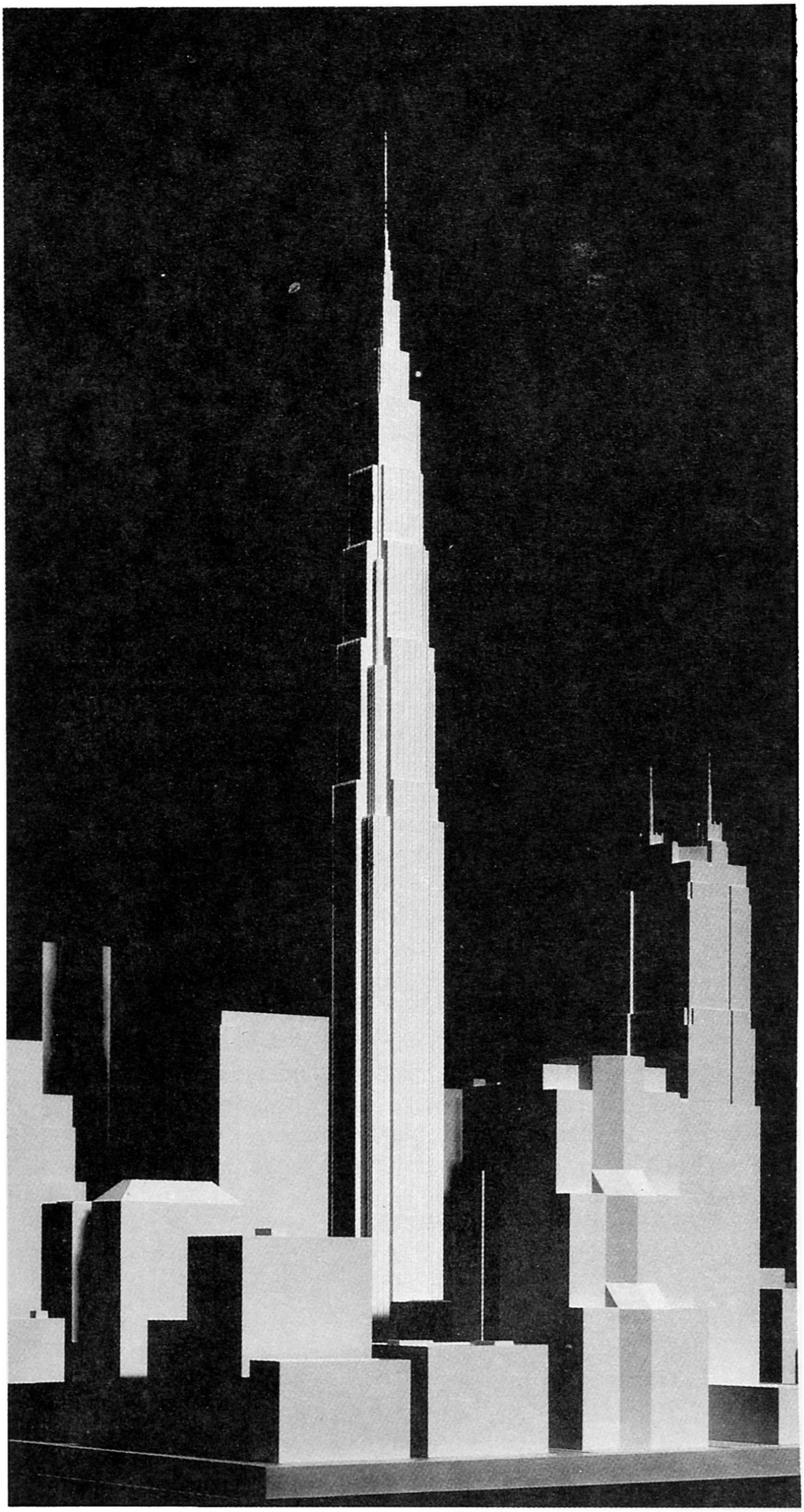
The caissons are tied together with a series of grade beams. Passive pressure on the edge of these lugs and on the projected side surfaces of the caissons provides the lateral base shear resistance for the Miglin-Beitler Tower.

Columns outside of the footprint of the main tower, supporting the parking levels in the bustle of the building, will be in turn supported on belled caissons. The belled caissons with maximum bell diameters of 3.7 metre will bear on a strata of hard pan material approximately 20 metre below grade level. The hard pan has an allowable presumptive bearing pressure of 2 MPa.

5. DIFFERENTIAL SHORTENING

Due to elastic strains, creep and shrinkage, the core strain rate is significantly greater than that of the perimeter fins. A computer program has been developed to analyze the need for vertical camber in the fins and core due to the differential shortening that will occur, primarily, during construction. A system of daily measurements during construction will provide input to constantly update the analysis results and provide information to the contractor.

The program will consider the bending stresses imposed on the link beams due to differential shortening between the core and fins, as well as the subsequent effect of relaxation in the link beams due to creep attributed to flexural compressive stresses.



6. DIFFERENTIAL TEMPERATURE

Portions of the exterior concrete fin columns are outside of the building window line and the controlled environment. To minimize differential temperatures, which can cause significant thermal stresses in the fin columns, the fins will be clad with an insulated exterior facade wall system. This system was designed to minimize thermal differences between fin columns and the core. Stack effect considerations in the cavity have been included.

7. STEEL VIERENDEEL TRUSSES

On each of the four faces of the building, steel vierendeels are employed to frame the 18.6 metre clear opening between the fin columns. The vierendeels consist of a 914 mm deep horizontal beam at each level with two 914 mm deep verticals. To eliminate stresses produced by creep and shrinkage strains in the concrete fin columns, the verticals in each vierendeel are provided with vertical slip connections. This has an added benefit of channeling all of the gravity loads on each of the building faces out to the fin columns to help eliminate uplift forces on the foundations. The steel face vierendeels are to be shop fabricated as horizontal trees 3.8 metre tall by 18.3 metre long. Field connections are simple bolted connections. This system allows for all of the welded connections to be shop fabricated resulting in an economical and elegant solution.

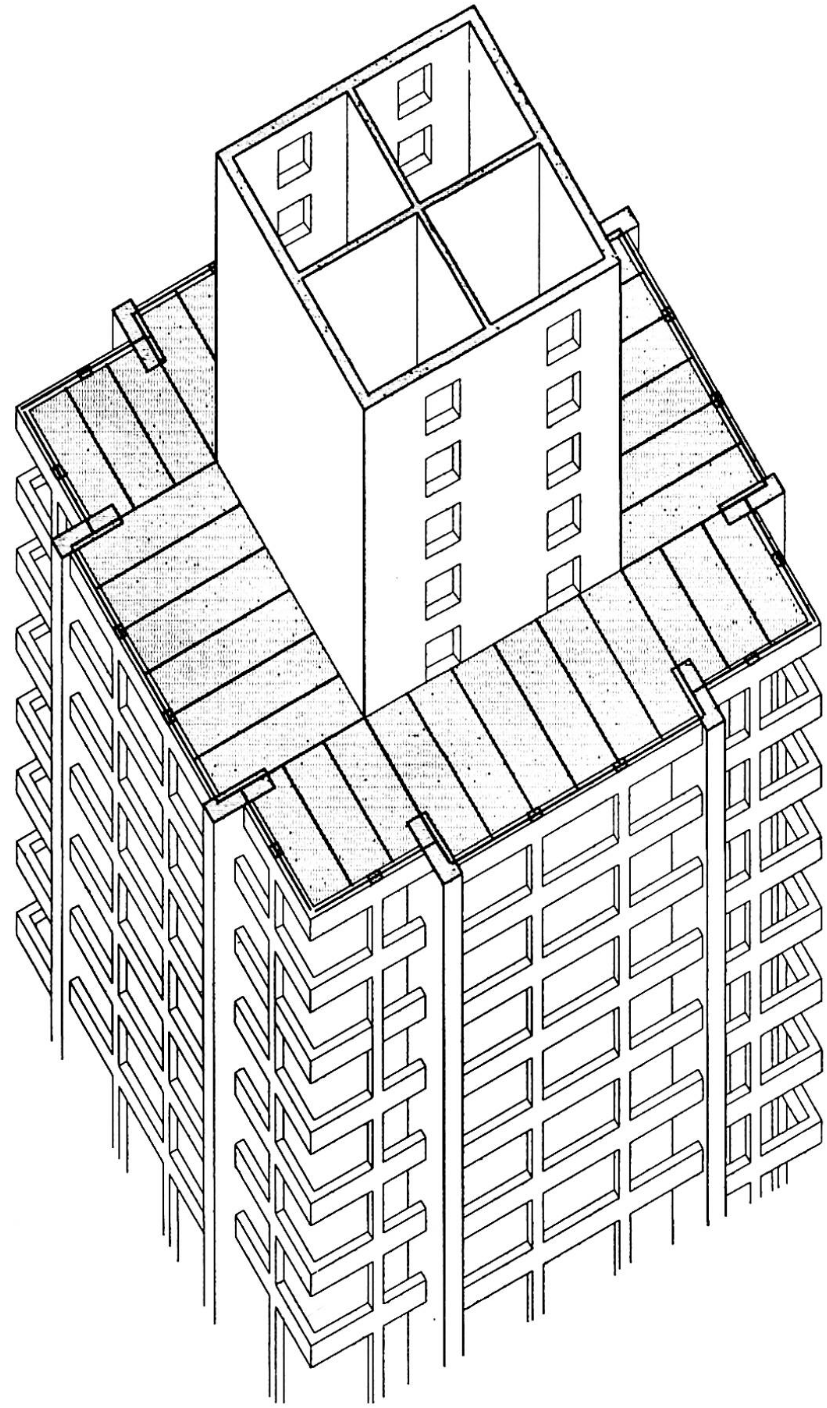
At each of the corners of the Miglin-Beitler Tower, the floor slabs protrude beyond the fin columns by up to 8 metres. Again, it was desirable to channel the gravity loads from these areas to the fin columns to help eliminate uplift in the foundations due to lateral loads. An added challenge was to frame the corners without having a vertical element at the corner, thus allowing the corner offices unobstructed views. The solution to this problem was a vierendeel truss, similar to the face vierendeels. The corner vierendeels are to consist of a horizontal steel beam at each level with a vertical steel beam at the center of each face, thus allowing the corners of the building to be column free. Unlike the face vierendeels, all of the vertical connections are not slip connections. This is to allow the corner vierendeels to resist unbalanced floor loads.

8. STEEL SPIRE

The Miglin-Beitler Tower is to be topped with a 137 metre tall steel framed spire (see figure 4). The spire contains mechanical equipment, window washing equipment, antennas and communication systems. The main structural framing consists of 12 exterior columns that cascade out at each of the setback levels. Each level of the spire contains horizontal bracing that stabilizes the structure. In addition each of the elevations is typically x-braced. Topping off the spire is a section of 2.4 metre diameter steel tube. The tube is to be perforated with openings that allow for the installation of a wide range of broadcast equipment. The entire broadcast support structure is to be clad with a material that is transparent to the broadcast equipment.

9. CONSTRUCTION

The reinforced concrete core, fin columns and link beams will be simultaneously cast at each floor level using a specially developed gang form system. This procedure will eliminate the need for any steel erection columns and provide a three-day floor cycle.



MIGLIN-BEITLER TOWER

FIGURE 5 – Isometric View

Underslung cranes below the gang form will erect floor beams and exterior Vierendeel trusses approximately 3 floors below the level of completed concrete construction (see figure 5). This is made possible by the simple shear connections mentioned above between all steel and concrete elements.
The transfer walls at the three intermediate mechanical levels will be built later-on in the sequence. This not only facilitates erection speed but minimizes resistance to differential shortening between the core and fin columns.

Construction of the building will take approximately three years from start to finish. To increase the economic viability and profitability of the structure, a four part phased occupancy will be used. The first phase includes all of the parking in the lower levels of the structure. These levels will generate a significant income for the owners while construction of the tower is ongoing. The second phase of occupancy includes the lower office levels up to the sky lobby transfer level. This will allow the owner early rental of commercial space in the lower floors. The third phase of occupancy includes the remainder of the office levels up to the base of the spire. This phase will also include observation levels. The final phase of construction will include the steel framed spire housing the upper mechanical level, window washing and broadcast equipment. The phased occupancy approach to the construction of the Miglin-Beitler Tower was developed to enhance the economic feasibility of the project.

10. CONCLUSION

A cruciform tube structure provides a safe, elegant, efficient and constructible solution to the challenge of designing the world's tallest building, the Miglin-Beitler Tower. The proposed structural solution combines the erection speed of concrete construction, the flexibility for future change and the efficiency for horizontal spans of a steel floor system, and the superior dynamic acceleration response of a composite lateral load resisting structural system. The project team is listed in figure 6.

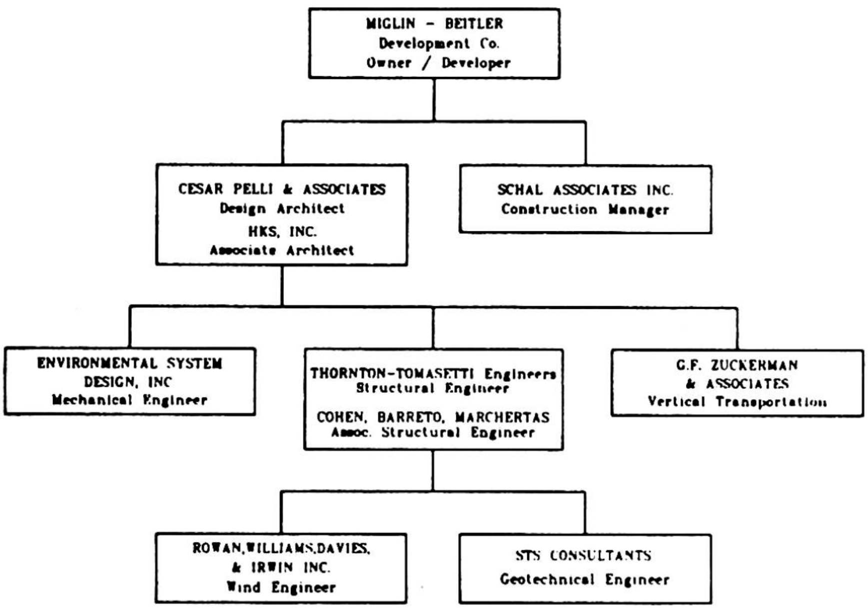


FIGURE 6 – Project team

High Rise System Concepts

Concepts pour les structures de maisons hautes

Konzepte für Hochhaus-Tragsysteme

Hal IYENGAR
Structural Engineer
Skidmore, Owings & Merrill
Chicago, IL, USA

Hal Iyengar is a Partner and Chief Structural Engineer of Skidmore, Owings & Merrill, Architects & Engineers, Chicago. He has received degrees from Universities in India and the University of Illinois, Urbana. Major projects include the Sears Tower and John Hancok Center, Chicago.

SUMMARY
This paper presents concepts for tall building structural systems developed since the introduction of the tubular system. The influence of changing aesthetic considerations and that of contextural relationships in the urban setting structural form and system is presented. Structural systems for multiple-use buildings and stacked atrium concepts are discussed together with newer structural systems including mixed steel-concrete and superframe structures. Some recent examples of exposed steel systems are also included.

RÉSUMÉ
L'étude présente divers concepts appliqués aux structures des maisons hautes depuis l'adoption des systèmes tubulaires. Elle rend compte de l'influence de l'évolution sur le plan esthétique et des relations contextuelles en milieu urbain entre forme et système de structure. Elle examine les systèmes de structure des bâtiments à usage polyvalent et les concepts d'atrium étagés ainsi que les systèmes de structure plus récents en construction mixte acier-béton et les concepts de super-cadres. sont également présentés. Des exemples récents de systèmes d'ossature apparente en acier sont également presentés..

ZUSAMMENFASSUNG
Das Referat befaßt sich mit Konzepten von Tragsystemen für Hochhäuser seit Einführung des Rohrsystems. Es bespricht den Einfluß sich ändernder ästhetischer Erwägungen und kontextueller Beziehungen bei der städtebaulichen Planung auf Bauform und Bausystem. Darüber hinaus werden Bausysteme für Mehrzweckgebäude und übereinander angeordnete Atriumkonzepte sowie neuere Baukonzepte wie Mischformen aus Stahl und Beton und der "Superrahmen" diskutiert. Das Referat führt ebenfalls einige neuere Beispiele von sichtbaren Stahltragwerken an.

1. INTRODUCTION

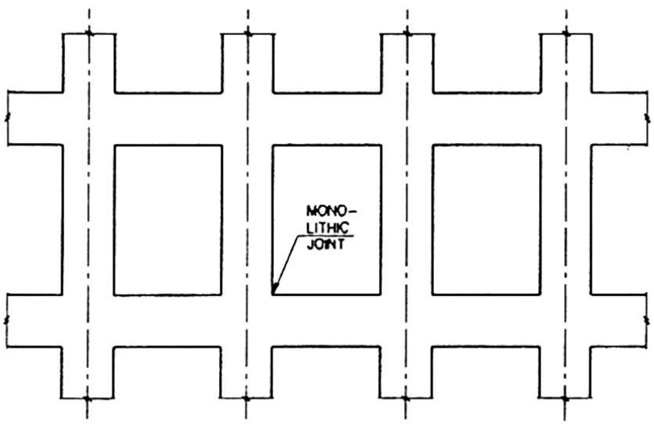
The structural systems for tall buildings have undergone a revolutionary process since the introduction of the tubular system. The rectilinear prismatic ideal of the 1950's and 1960's has been replaced by shaped, non-prismatic forms, mainly to respond to site geometry, urban planning issues and the visual impact of the varying vertical profile. The tubular system has been very adaptable to such changes and various refinements, such as the bundled tube and clustered tube, have responded well to such shape modulations. Mixed steel-concrete systems, particularly the composite tubular system, have further enhanced the application to such shaped forms. New architectural concepts are emerging, which incorporate internal and stacked atriums. New structural systems, such as the "superframe" are being developed to respond to the needed space flexibility. This paper will examine these developments on a conceptual basis.

2. TUBULAR SYSTEM

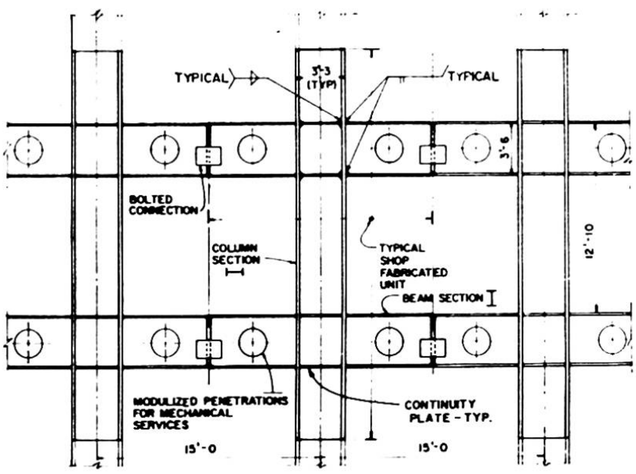
The development of the equivalent cantilever tubular system in the mid-1960's represented a significant milestone in the evolution of tall building systems. Earlier structural systems placed the lateral load resisting frame only in the interior in a plane frame arrangement in two directions. Some later versions introduced shear trusses or shear walls in the building core to provide some degree of stiffening by a vertical cantilever element. Partial tubular systems engaging the exterior frames with core trusses by means of belt and outrigger trusses also evolved. While all of these different improvements contributed to extend the range of application of frame type behavior, the radical departure occurred only when the structure was placed on the perimeter and was so interconnected as to act like a three-dimensional cantilever utilizing the entire exterior form. The characteristics of this exterior structure were that of a wall, giving rise to the terminology "tube structure" to designate the silo-like cantilever behavior of this structure. The material most readily adaptable to create the wall-like structure was concrete where wide columns and deep beams were cast monolithically (Fig. 1a) in a closely spaced formation. The adaptation of the exterior tube frame to structural steel required considerable welding, which in a closely spaced column format, was not cost effective. However, the development of a shop fabricated unit (Fig. 1b), where all joint welding can be done in the shop in a horizontal position; the unit then transported and field bolted, allowed this principle to be applied to the steel. The trussed tube involving exterior diagonalization also evolved as a unique solution utilizing structural steel. Fascia diagonals are interconnected between spandrels and columns to form a truss on each building facade. (Fig. 1c) This produced equivalent cantilever behavior similar to that of the framed tube and in fact, was more efficient because of reduced shear lag. The vocabulary of the exterior tube has been well established both in steel and concrete and a wide variety of buildings from 30 to 110 stories using this system have been constructed.

3. THE SHAPING OF EXTERIOR TUBES

The contextual relationship to the urban grid and massing with respect to neighboring buildings, as well as the aesthetics of a non-prismatic form, are all factors which impact the form and shape of buildings. The rigidly organized bay frame vocabulary allowed little freedom for such shaping without drastically reducing structural efficiency. The exterior tube by its very nature can allow considerable latitude in shaping. The basic requirements are that the tube structure be continuous and of a closed form on the exterior of the building. The overall depth and width of the shape, the degree of asymmetry and the height-width ratio all affect the efficiency of the form.



(a) Concrete Framed Tube Wall



(b) Steel Framed Tube

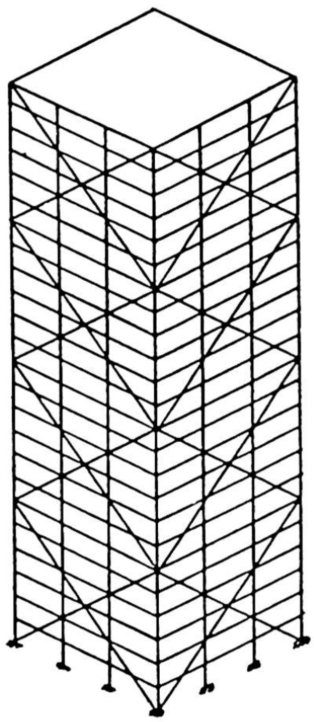


Fig. 1 Exterior Framed Tubes

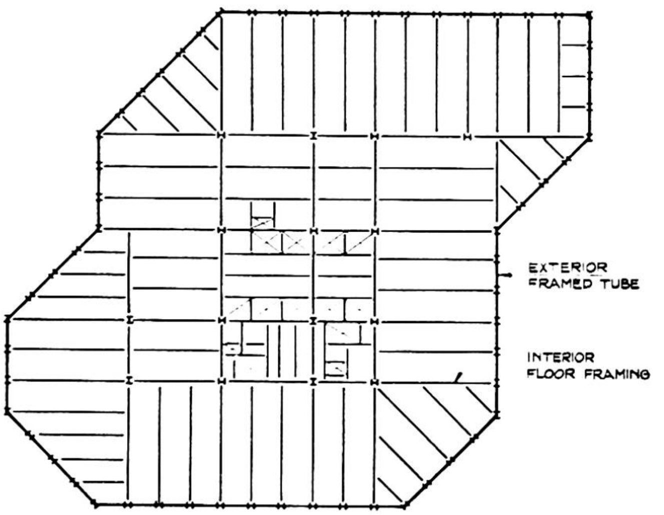
The Sixty State Street Project, Boston, Massachusetts (Fig. 2a) is a 45-story office tower which was configured as practically a free form to preserve sight lines from neighboring tall buildings. An exterior steel framed-tube was used with columns at 3m on center with a prefabricated "tree" type erection unit.

The 45-story Madison Plaza Tower, Chicago, Illinois (Fig. 2b) was conceived as a square shape with one corner truncated providing a series of steps along the diagonal face. This allows an open plaza at the corner in this tight urban site, as well as providing a crystalline, glazed, stepped facade for vistas to Lake Michigan and the City. The exterior line is taken along the linear diagonal rather than a stepped one to preserve the efficiency of the tube. 750mm and 900mm deep rolled beam sections were used as columns with the typical erection unit.

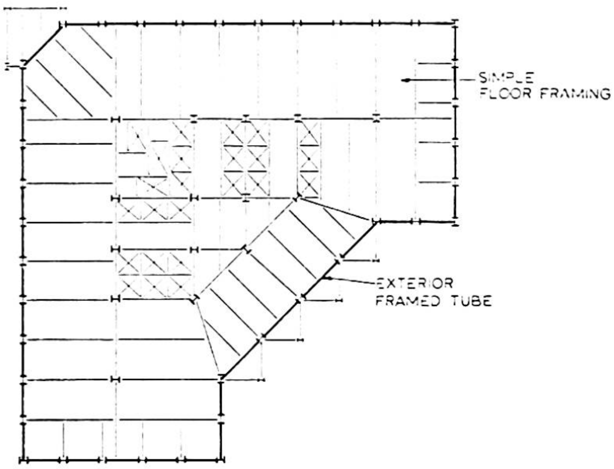
4. THE BUNDLED TUBE

The concept of bundled tubes was introduced with the Sears Tower, Chicago, Illinois. (Fig. 3a) The need for vertical massing variation in a modular fashion created the idea of bundling smaller size tubes which can rise to different heights. The structural efficiency of the overall form is greatly enhanced because of the presence of the interior frame lines which reduce the shear lag effect of a pure exterior tube.

The modularity and the conceptual basis of the bundled tube have a broad application. The cells or tubes can be arranged in a variety of ways to create different massing. It can be applied to 30 stories as well as ultra-tall structures. Further, the shape of each tube itself can be changed to any other closed clustering shape. The concept of bundling is equally applicable to both steel and concrete.



(a) Sixty State Street,
Boston, Massachusetts



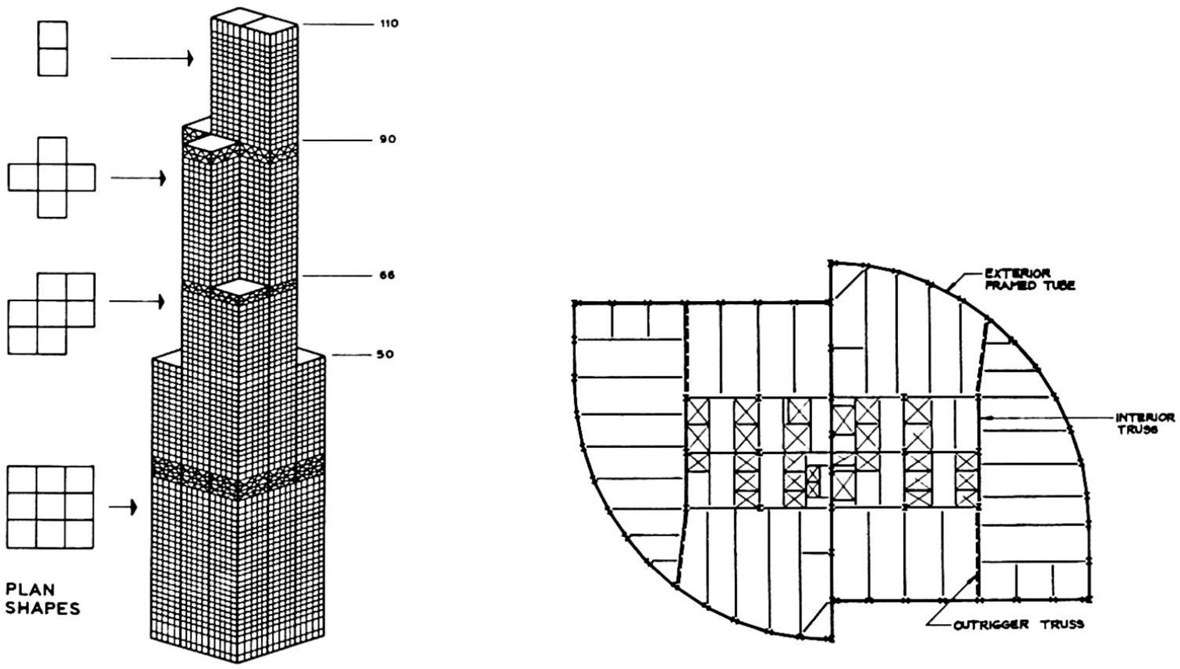
(b) Madison Plaza,
Chicago, Illinois

Fig. 2 Shaped Tubular Buildings

A recent building derived from the Sears Tower system is the Allied Bank Building, Houston, Texas, which is another example of the bundled framed tube application. The shape is formed by two quarter circles placed anti-symmetrically about the middle tubular line. See Fig. 3b. The column spacings are 4.5m with the usual "tree" type construction. The system also uses two vertical trusses in the core which are connected to the exterior tube by outrigger and belt trusses. Significant improvement in tubular behavior is obtained because of the participation of the trusses in reducing shear lag.

5. MIXED STEEL-CONCRETE SYSTEMS

Mixed steel-concrete [1] systems have been established and are now used as readily as either all steel or all concrete systems for high-rise buildings. A wide variety of mixed forms are generally applicable, such as the composite tubular and the concrete core braced systems. The properties of concrete that are most attractive are its rigidity and its ability to be cast into different types of structural elements. Therefore, most mixed system compositions rely on concrete for lateral load resistance. Shear wall elements and/or punched wall or framed-tube elements with monolithically cast beam-column joints are primary elements used for lateral load resistance. Steel floor faming in mixed systems is advantageous because of the ability to span longer distances with lighter members and make possible larger column free space.



(a) Sears Tower, Chicago, Illinois (b) Allied Bank, Houston, Texas

Fig. 3 Bundled Tubes

The First Canadian Centre, Calgary, Canada consists of two towers and a 10-story banking pavilion, located in an L-shaped site in downtown Calgary. The two towers are 64 and 43 stories. A sculpted form which provides diagonal vistas to the mountains and city was highly desirable for this prominent corner site. Each tower is similarly shaped, basically involving a parallelogram with truncated and re-entrant corners. The structure is based on a tube-in-tube concept involving an exterior reinforced concrete framed-tube and an interior shear wall core tube. Structural steel floor framing and other interior steel columns complete the system, as shown in Fig. 4.

A recent trend in exterior architecture has been to express facade steps, protruding triangular bays and other facade profile modulations. These demand a lighter structure on the exterior which can be provided in structural steel. In these instances, a structure that is concentrated in the core of a building for wind resistance will offer flexibility of framing on the exterior. A logical mixed combination here is a concrete shear wall core which resists all wind forces surrounded by simple steel framing for floors and exterior columns.

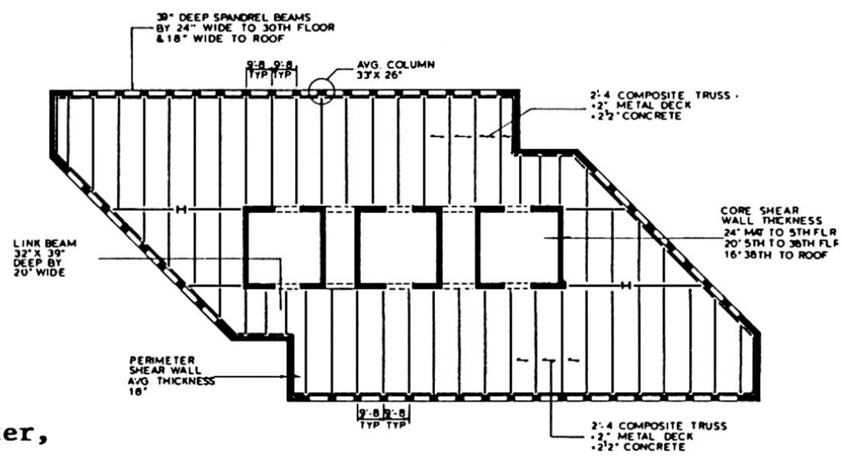


Fig. 4 First Canadian Center, Calgary, Canada

Fig. 5 shows an example of a 44-story core braced system which was configured to fit an unusual site. The core is augmented by fascia moment frames.

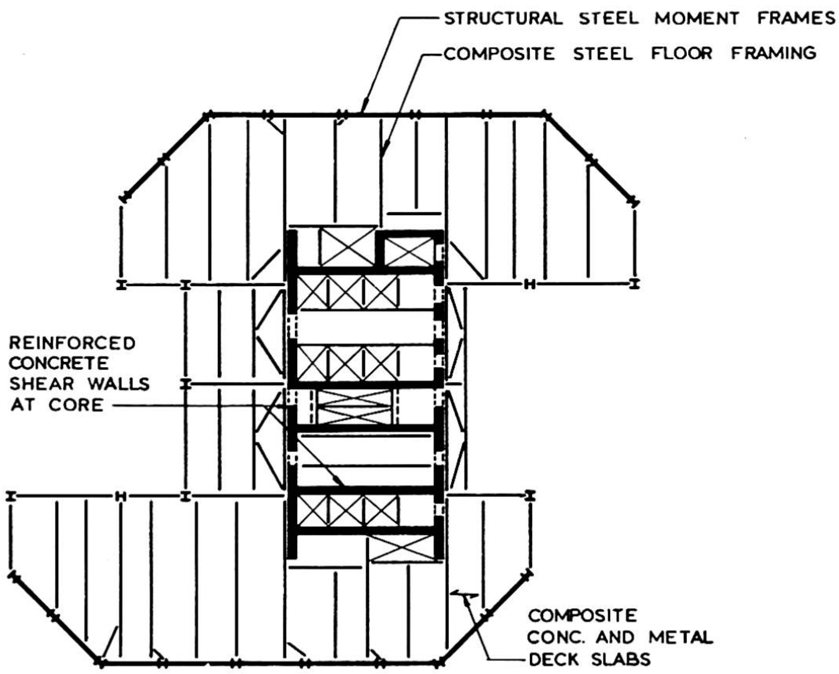


Fig. 5 Tower 49, New York, New York

6. MULTIPLE-USE TOWERS

Multiple use towers, which incorporate vertical stacking of different occupancies such as parking, commercial, office and residential, have become attractive to address concerns about the quality of inner-city life and better utilization of energy resources. The functional planning of spaces often influences the choice of a structural system. [2] In terms of floor spaces, office and commercial buildings use longer-span structural systems consistent with the space requirements, whereas housing and apartment buildings use relatively shorter span structural systems consistent with residential room sizes. Although both reinforced concrete and structural steel are used for office buildings as well as residential buildings, the structural systems are quire different. The office building requires longer spans as well as much more complicated mechanical and electrical systems, almost invariably using false ceilings, whereas residential buildings with less complicated mechanical and electrical systems, do not require the use of false ceilings except for special cases. Flat-plate, reinforced concrete slab construction has therefore become the most accepted floor system for residential buildings whereas beams, joists or grid beam (waffle) systems are used more frequently for office and commercial buildings. The generally desired stacking of residential over office, and office over commercial, and commercial over parking, brings about a form which needs to be wider at the lower floors and narrower at the top to create the effective lease spans for different occupancies. A gradual reduction of the span from the top to bottom makes the overall structural system continuous on the exterior, rather than a wedding cake type arrangement which results in discontinuous, incompatible structural pieces.

A recent example of a multiple-use tower in Chicago is that of the 64-story Olympia Center. The program involved 3 floors of below-grade parking, 6 floors of department store, 17 floors of office and 40 stories of apartments. The floor size of 36m x 52m at the bottom varies to 18m x 52m at the apartment levels. The building is transitioned from the 15th to the 30th floors by a continuous curve, as shown in Fig. 6. The moldability of the concrete makes such a curvature possible. Another aspect of a commercial-office-apartment combination is the integration of the facade fenestration requirements in the same system. Commercial space requires very little window fenestration, office space requires regular fenestrations reflecting the modular nature of the office space and apartments require combinations of opaque and window spaces reflecting requirements of different rooms. In addition, in the Olympia Center, two-story duplex apartments were desired which required removal of the spandrel beams over the living room width. The bearing wall nature of the framed tube permits different degrees of openness desired for different occupancies. A study of the building structural elevation reveals more solid portions in the commercial area, a consistent framed-tube grid in the office areas and a more flexible grid which begins to open up at the apartment levels with duplexes in the middle of the faces, eventually growing out to the corner areas and finally turning into a bay frame structure at the top few floors. The floor framing shows a wide pan joist type of framing for the longer span office framing and flat plate framing for the apartments.

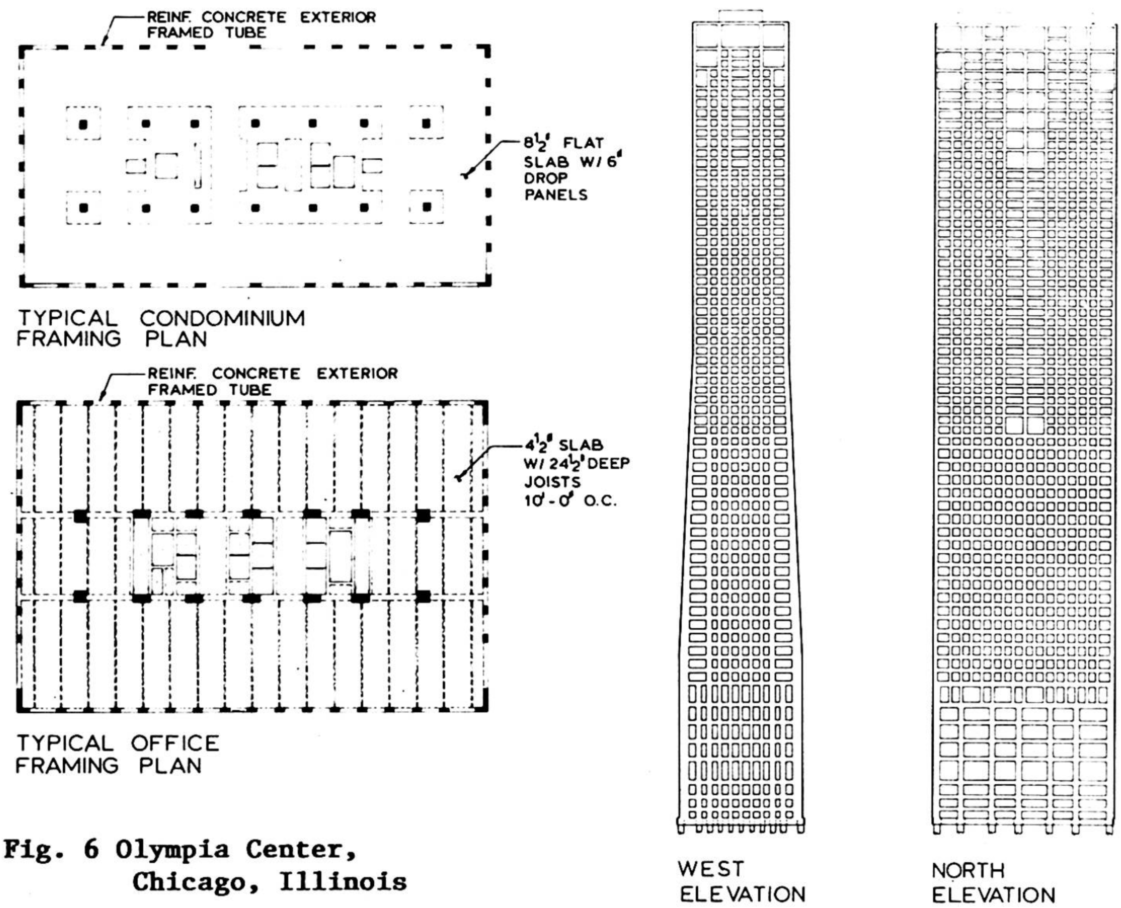


Fig. 6 Olympia Center, Chicago, Illinois

The 57-story One Magnificent Mile Building, Chicago, Illinois [2] typifies another approach to achieving vertical modulation of spaces, that of bundling or clustering of different tubes. The free-form structure is composed of three, near hexagonal, reinforced concrete framed tubes with the highest tube at 57 stories and the others at 49 and 22 stories each. (Fig. 7) The arrangement of tubes and their orientation was determined from the site configuration and optimization of vistas to Lake Michigan. The clustering principle was highly useful in molding the overall form around this L-shaped site with a diagonal frontage. The hexagonal shape for each tube created a highly faceted format for the overall architectural form. The lower 20 stories, which include all three tubes, are occupied by commercial and office space with the rest devoted to apartments. It should be noted that the two and single modules are especially suited for apartment layouts. The floor system is a reinforced concrete, flat slab system in both office and apartment floors. The structure for the tubular lines involves columns at close centers and deep spandrel beams.

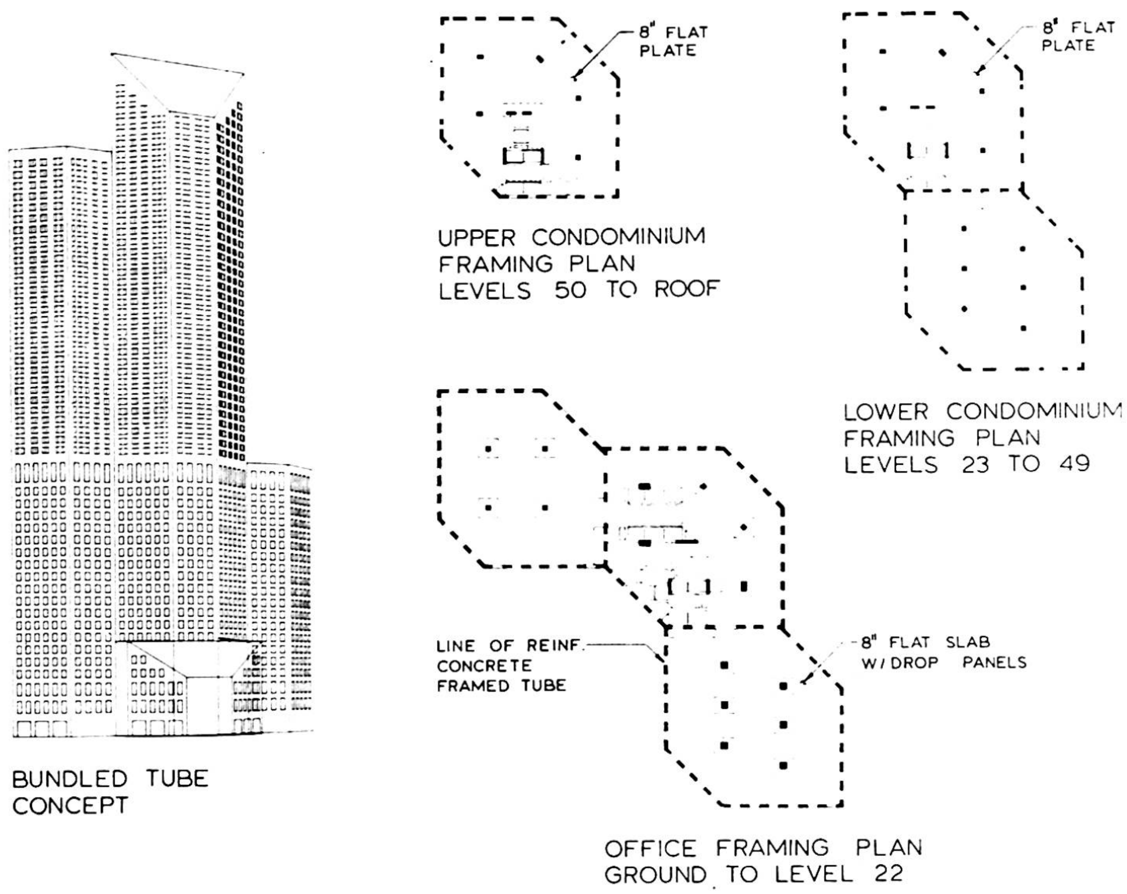


Fig. 7 One Magnificent Mile, Chicago, Illinois

7. SUPERFRAME SYSTEM

Superframes are megaframes in the form of a portal which are provided on the exterior of a building. The portal frame of the superframe is composed of vertical legs in each corner of a building which are linked by horizontal elements once in about 12 to 14 floors. Since the vertical elements are concentrated in the corner areas of a building, maximum efficiency is obtained for resisting wind forces. The vertical legs and the horizontal links are themselves frames with large dimensions in the plane of the frame.

The concept of the stacked interior atrium evolved from energy conservation ideas relating to the amount of the exterior facade exposed to the outside environment. A volume of squatter form with large floor plates will reduce the proportion of window area to the floor area. [3] However, large quantities of interior spaces are not optimum for prime office rental. The object of interior atriums is to remove interior volumes of spaces that are considered undesirable and provide for exciting three-dimensional sheltered spaces. The window wall at the atrium would then be considered equivalent to an exterior wall and thus, improves the quality of prime spaces. These atriums can be shaped in a way to provide a variety of floor configurations and sizes. The vocabulary of stacked atriums and the vertical organization is shown in Fig. 9. Full floors separate these atriums and the formations of portal openings on the facade express these atriums and also allow for access to exterior light without heavy structural encumbrance. Superframes are then equivalent cantilever forms which integrate this principle of stacked atriums. In order to maximize the cantilever efficiency, the members of the portal frame, namely, the verticals and horizontals, will have to be considerably stiff in their own plane. Fig. 8 shows two steel possibilities and a composite variety where the superframe elements are made of reinforced concrete. In Fig. 8a, a broad diagonalization like that of a trussed tube is used for the elements. In Fig. 8b, a more delicate diagrid bracing is provided to the elements. The difference is in the possibilities for architectural expression. Superframes can also be conceived in a composite form, where the vertical legs of the superframe are punched wall-like elements which are interlinked by steel trusses. The interior framing would then be in structural steel as in the composite tubular system. The rigidity and moldability of concrete makes possible these punched walls or framed tube elements to develop an overall equivalent cantilever when properly interlinked. See. Fig. 8c.

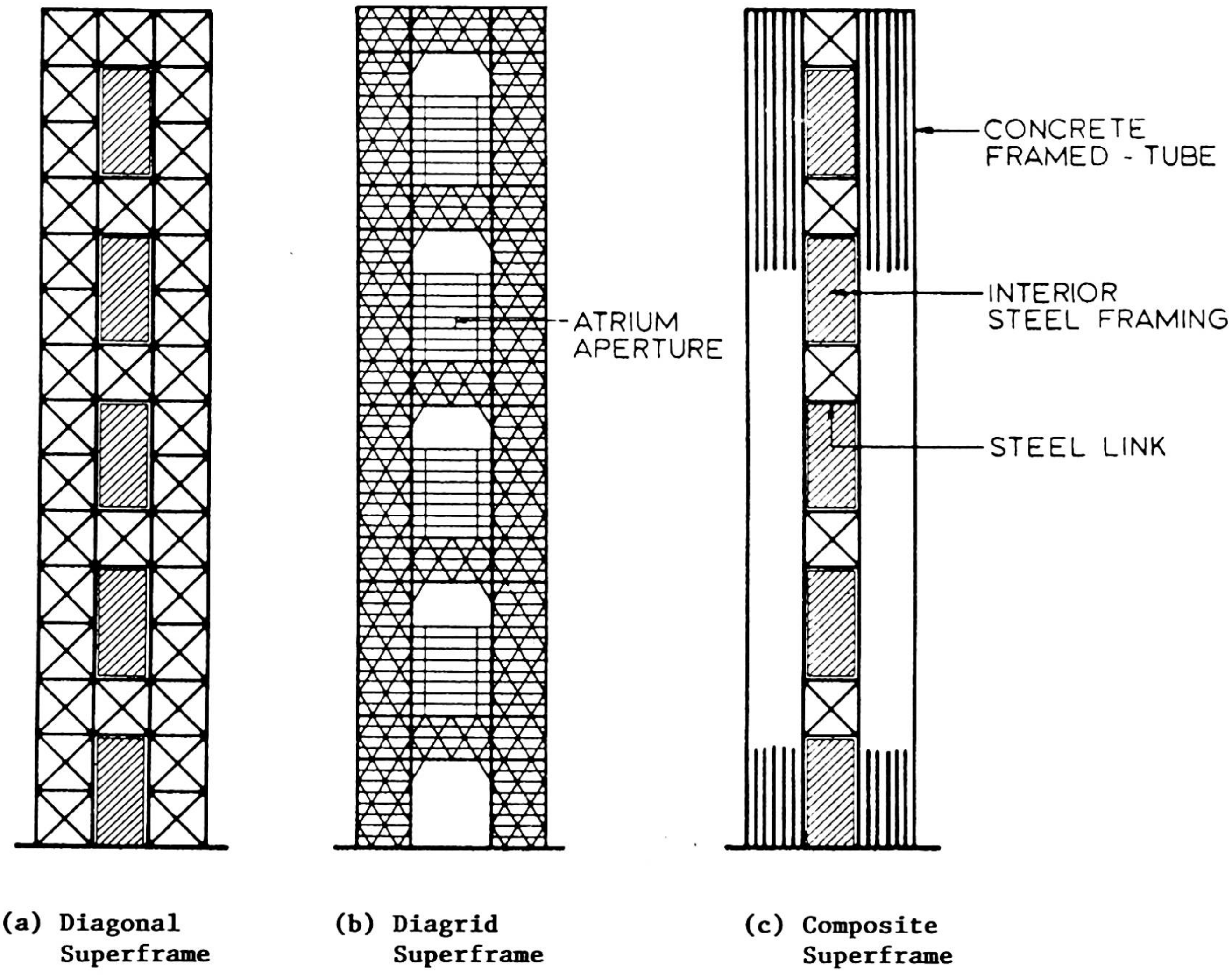


Fig. 8 Superframe Concepts

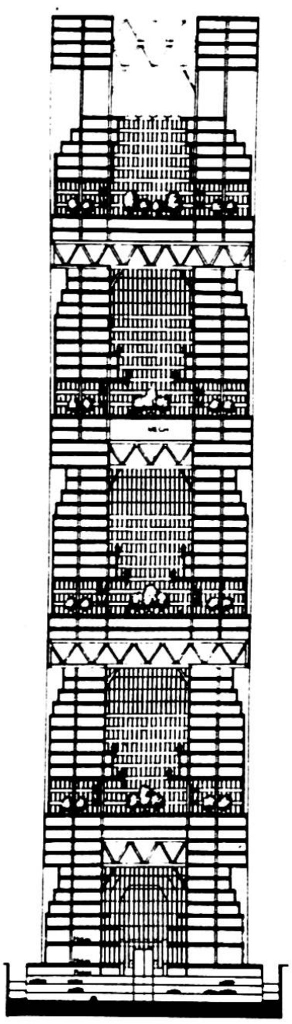


Fig. 9
Stacked
Atria-
Superframe
Concept

8. EXPOSED STEEL SYSTEMS

The beauty and essence of exposed structural steel can form the basis for architecture in multi-story buildings. The exposed steel frame may range from simple beam-column expressions to more intricate arrangements of trusses, arches or other exotic forms. From an architectural point of view, a clearly articulated structure on the exterior is desired. This articulation is characterized by the crisp proportions of steel I-beams, columns, and built-up members, and the honest expression of the connecting joints both bolted and welded. Structural functionalities, such as primary compression or tension elements or joint load-carrying mechanisms can be appropriately expressed and proportioned. The architectural aesthetic is based on clearly defining open web-like forms to allow the play of daylight through the structure. This must be balanced by the need for robustness and structural integrity, particularly at the member joints.

The issues of corrosion and fire protection must be addressed in engineering exterior exposed steel buildings. Improvements in corrosion protection through the development of durable, long-life, fluorocarbon paint systems have enhanced exposed steel construction. The use of state-of-the-art fire engineering concepts in designing exposed steel frames has gained momentum. Analytical approaches to determine the steel temperatures when exposed to different fires, as well as the determination of the character and nature of the fires, are now well documented and accepted.

A recent example of an exposed steel system is shown in Fig. 10. [4] The Broadgate Project is a major office development on the northeast edge of the City of London, located adjacent to the Liverpool Street Train Station. The railroad tracks cover most of the site and must be left intact and usable, thus requiring that the project be built to span over a large portion of the tracks, supported on columns only where permitted by the track and platform layout. The main block of Exchange House is a 10-story office building, 78m x 54m in plan, supported on four (4) segmented, tied, parabolic arches that span the 78m over the railroad tracks below. The two exterior arches, their ties and the columns and beams that frame into them are located so as to create a 2m wide gallery at the perimeter creating a structural expression for the building.

The architectural form, expression and articulation are all based on the exposed, painted steel structure. The building enclosure forms a smooth metal and glass skin background to enhance the clarity of the structure. Member proportions and joint details follow strict structural logic to express directly the functions and workings of the structure. (Fig. 10)

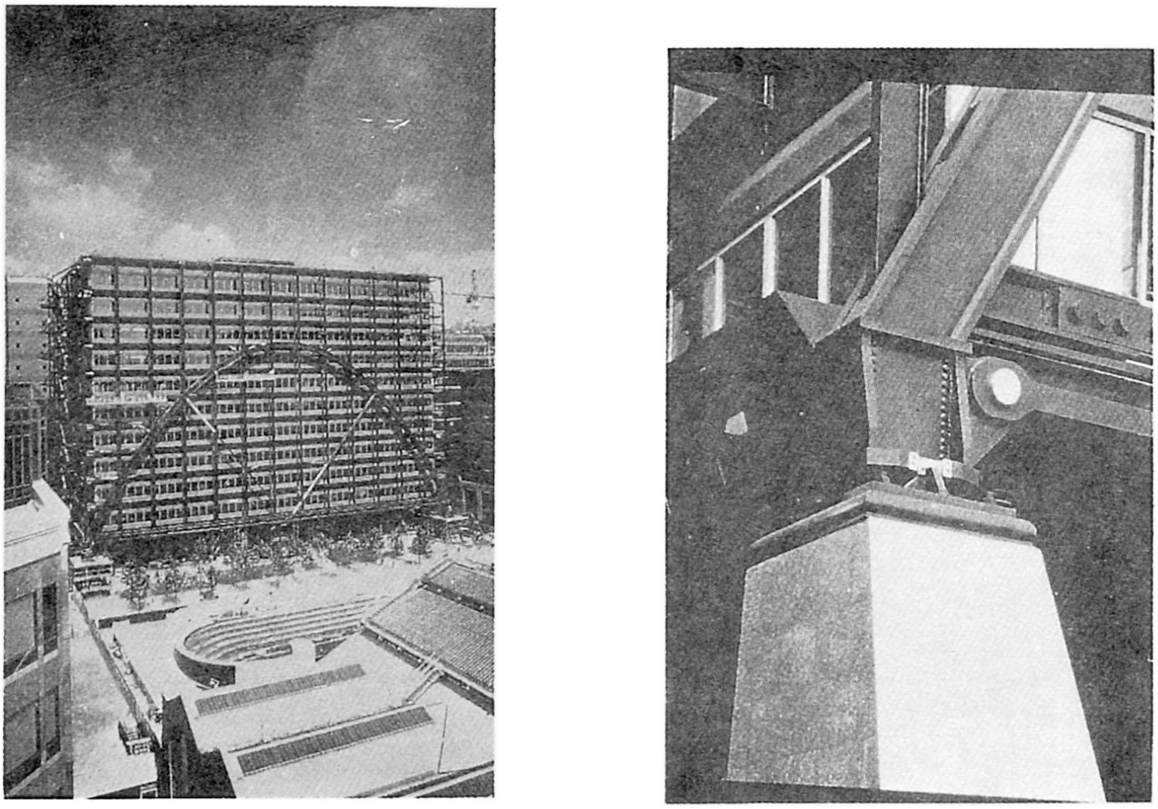


Fig. 10 Exchange House, Broadgate Project, London, England

Hotel Vila Olimpica is a hotel, residential, commercial and office development on the waterfront in Barcelona, Spain. [5] Under construction now in preparation for the 1992 Olympics, the complex consists of a 6-story office block, a low-rise retail and public space, and a 44-story hotel-apartment tower. The apartment tower, the most prominent element in the composition, has an X-braced steel frame pulled away from the exterior curtain wall. The form of the exterior structure and the articulation of its connections, together with the shadows it casts upon the metal-and-glass curtain wall, establish the primary architectural character of the tower. (Fig. 11)

The hotel tower is square in plan, roughly 33 meters on a side, with its steel framework placed away from the curtain wall an additional 1.5 meters. Each side of the steel frame has two X-braced bays flanking a center bay of moment-connected spandrel beams. Additional X-bracing at the bottom, middle and top of the center bay tie the two X-braced frames together so they act as a unit.

The tower's interior layout of individual, compartmentalized hotel rooms gave the structural designers an opportunity to create an external steel framework that is completely free of fireproofing. Fire engineering and high-temperature structural analysis showed that the steel structure will remain safe when exposed to the most severe fire that can be expected from such a compartmentalized floor plan.

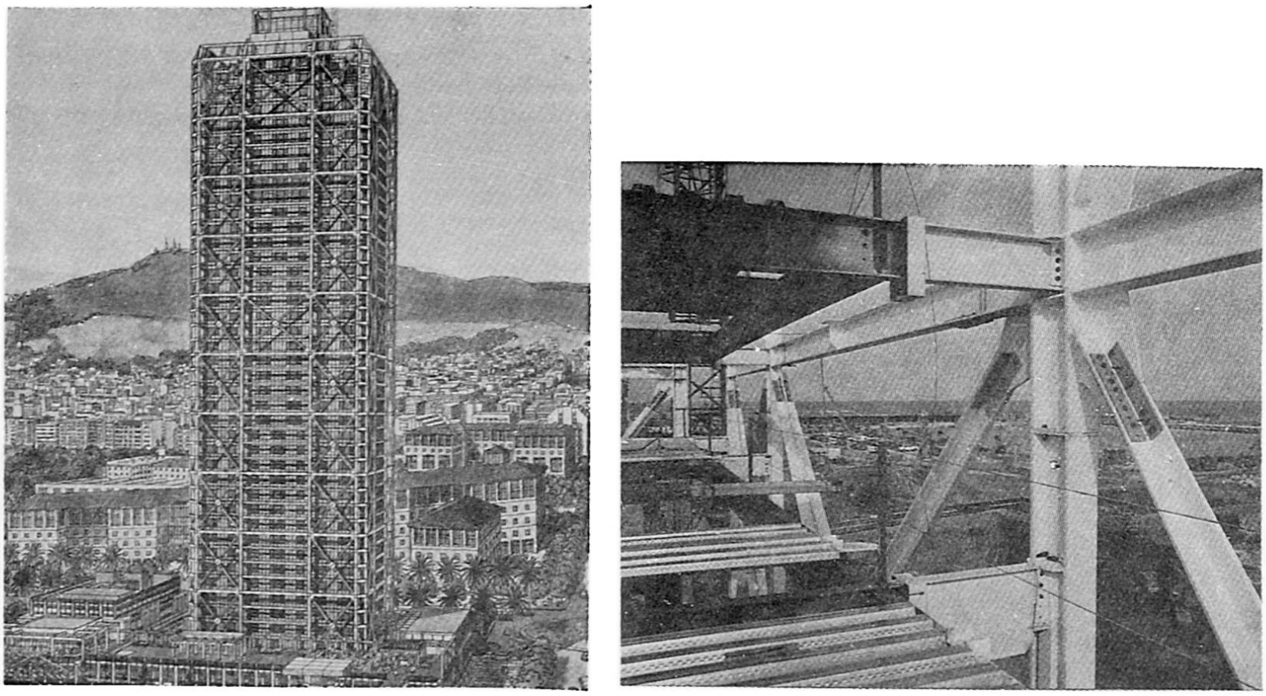


Fig. 11 Hotel Vila Olimpica, Barcelona, Spain

9. CONCLUSIONS

Structural systems for tall buildings will continue to be influenced by building form and urban massing considerations. The principle of an equivalent cantilever as demonstrated by the tubular system will continue to be dominant. The bundled tube and clustered tube system will provide some degree of needed flexibility regarding massing and functional integration. Newer structural forms which integrate with special concepts, such as the superframe, will further evolve. Future structural systems will often borrow elements from previous systems if they can be utilized efficiently and a combination is derived to suit the needs of a certain project. Combinations may involve framed tubes, bundled tubes, trussed tubes, mixed systems, superframes, etc. The ability to analyze any three-dimensional structure on the computer readily and economically and verify its behavior and efficiency will permit such systems methodology to flourish.

REFERENCES

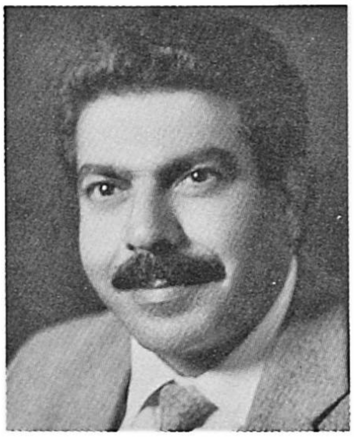
1. IYENGAR, HAL, Recent Developments in Mixed Steel-Concrete Systems. United States/Japan Seminar on Composite construction, University of Washington, Seattle, Washington, July, 1984
2. KHAN, FAZLUR R. and EL NIMEIRI, MAHJOUB M., Structural Systems for Multi-Use High-Rise Buildings. Council on Tall Buildings and Urban Habitat, Developments in Tall Buildings, 1983, Chapter on Structural Systems (SC-1), pp. 221-235
3. IYENGAR, HAL, Steel Systems for High-Rise Buildings. International Conference on Steel Structures. Singapore, March 7-9, 1984
4. IYENGAR, HAL, Exposed Steel Frame-A Unique Solution for Broadgate, London. AISC Conference Proceedings, AISC National Steel Construction Conference, Nashville, Tennessee, June 22-24, 1989
5. IYENGAR, HAL and BLETTE, KARL, Graphic Interaction in Structural Design. Third International Symposium on Computer Aided Design in Architecture and Civil Engineering-ARECDAO 91, Barcelona, Spain, April 10-12, 1991

Analysis of Framed Buildings Having Arbitrary Wall Panels

Calcul des ossatures en portique à voiles raidisseurs

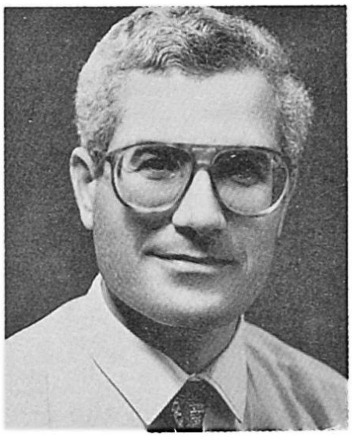
Berechnung von Rahmentragwerken mit Schubwänden

Musa RESHEIDAT
Assoc. Prof.
Univ. of Science & Technology
Irbid, Jordan



Musa Resheidat, born 1946, graduated with B.Sc. (1969), M.Sc. (1978), Ph.D. (1980) in civil engineering from Cairo, Stanford, and Purdue Universities. He has both academic and professional experience for over 22 years, has over 30 technical publications. He is now the "Engineer" for $200 million projects.

Jamal UMARY
Civil Engineer
Irbid, Jordan



Jamal Umary, born 1956, with B.Sc (1981) from AUB and M.Sc. (1990) from JUST. He has professional experience during his work as a site engineer for two major projects in Jordan. He is now a self-employed practising civil engineer.

SUMMARY
The direct stiffness method is employed to analyze framed buildings having arbitrary wall panels as structural elements modeled by finite elements. A computer program is developed to handle the mathematical formulations, solution of equations and calculation of joint displacements and member-end forces. Example problem is presented to demonstrate the effect of wall panel configuration on the stiffness of the structure.

RÉSUMÉ
Basé sur la méthode des éléments finis, un logiciel pour ordinateur a été élaboré en vue de calculer les ossatures en portiques comportant des voiles raidisseurs de position arbitraire. La résolution du système d'équations des éléments porteurs fournit les déplacements des noeuds et les efforts intérieurs. L'article présente un exemple qui montre l'influence de la disposition des voiles sur la rigidité de l'ouvrage.

ZUSAMMENFASSUNG
Für die Berechnung von Rahmentragwerken mit beliebiger Stellung aussteifender Wände wurde ein Computerprogramm entwickelt, das auf der Methode der Finiten Elemente basiert (Weggrössenverfahren). Aus der Lösung des Gleichungssystems der Tragelemente ermittelt es die Knotenverschiebungen und Schnittkräfte. Das vorgestellte Beispiel zeigt den Einfluss der Wandanordnung auf die Bauwerkssteifigkeit.

1. INTRODUCTION

The majority of multistorey buildings are commonly used for housing, offices, and other public or commercial facilities. The classical structural system consists of frames including column and beam elements. Such buildings have slabs, shear walls, and wall panels or partitions which have been constructed to satisfy functional requirements. Designer tend to consider the infill panels as nonstructural elements loaded on the beam elements and hence treat the frames as conventional ones; a practice is adapted, but it is far from reality.

Recently, there has been an increasing recognition among structural engineers that wall panels are no longer considered as nonstructural elements, but they affect the structural characteristics of the building and may alter the structural behavior by increasing the stiffness of the structural system. This fact stimulated researchers to develop methods of analysis which simulate the actual behavior by incorporating the wall panels as structural elements. Each method is recognized by the manner of modeling the panels.

The concept of equivalent truss member to idealize the panels is used by one group of investigators [1,2,3] while another group modeled the panel by finite elements [4,5]. The third group focused on either experimental evidence [6] or rationalized the design approach [7]. The literature survey on these methods could be found in Ref.5.

This paper presents the two-dimensional analysis of framed buildings having arbitrary wall panels aiming at studying the effect of such panels to improve the stiffness of the structure in general and to minimize the lateral drift in particular. In addition to that, selection of arbitrary wall panels is sought amongst various configurations to optimize the structural behavior of the entire building. The study presented herein is a continuation of previous studies [8,9]. The structural system consists of three media, namely: the superstructure, the substructure and the supporting soil medium. For the superstructure, the stiffness method is employed to model the beam and column elements. The wall panels are modeled by plane stress finite elements. The substructure is modeled by uniaxial finite elements. The soil is modeled by elastic half space where the stiffness matrix is obtained by the inversion of the soil flexibility matrix.

2. METHOD OF ANALYSIS

The direct stiffness method is utilized to develop the overall governing matrix equation in the form:

$$[K]\{D\} = \{Q\} \tag{1}$$

where

[K] is the overall structural stiffness matrix,
{D} is the vector of unknown joint displacements, and
{Q} is the vector of joint loads.

2.1 Stiffness Matrices

The governing matrix equation of a framed member can be written in the form [10]

$$[k]\{d\} = \{q\} \tag{2}$$

where

[k] = the global stiffness matrix of the member;
{d} = the member-end displacements, and
{q} = the member-end components.

The stiffness matrix of the soil medium as modeled by elastic half space can be obtained by inversion of the soil flexibility matrix which takes the form

$$[F] = [f_{ij}] \tag{3}$$

$$f_{ij} = \frac{1-\mu^2}{\mu\ E\ x_{ij}};\ \ i \neq j\ \ and\ \ f_{ij} = \frac{2(1-\mu^2)}{\pi a E}C_{ij};\ \ i = j \tag{4}$$

$$C_{ij} = ln[\alpha + \sqrt{1+\alpha^2}] + \alpha\ ln[\beta + \sqrt{1+\beta^2}] \tag{5}$$

$$\alpha = \frac{a}{b}; \beta = \frac{b}{a} \tag{6}$$

where a, b are the length and width of loaded element of soil, respectively.
E, μ are the modulus of elasticity and Poisson's ratio of soil, respectively.
$x_{ij} = x_j - x_i$
x_i = distance of node i from the origin of global coordinates.

The soil stiffness matrix is then obtained as

$$[S] = [F]^{-1} \tag{7}$$

The finite element technique is used to model the raft foundation of footings by using uniaxial bending elements. The wall panel is divided into a group of plane stress rectangular finite elements [11]. The stiffness matrix of the element can be written as:

$$[K]\ =\ a\ b\ t \int_{\xi=0}^{\xi=1} \int_{\eta=0}^{\eta=1} [B]^T\ [C]\ [B]\ d\xi\ d\eta \tag{8}$$

where

[B] = strain-displacement matrix; [C] = stress-strain matrix; a,b,t are the length, width and thickness of element, respectively and $\xi\ =\ x/a$; $\eta\ =\ y/b$

The stiffness matrix of the panel is formulated and condensed to correspond only to the four corners of the panel. Then the condensed matrix is partitioned into a system of (2x2) submatrices. Using equilibrium conditions and compatibility requirements, the contributing stiffnesses from wall panels, raft foundation, and soil are added to the appropriate joint positions to assemble the overall stiffness matrix.

2.2 Load Vector

Each joint is subjected to the resultant of external horizontal, vertical actions and moments to form the joint load vector [Q] of Eqn.1. The external loads can be either gravity loads, live loads and the wind or the equivalent seismic horizontal forces.

2.3 Solution of Equilibrium Equations

The governing matrix equation as given by Eqn.1 is solved by elimination techniques which proved to be more efficient, versatile and reliable because the stiffness matrix of the entire structure is always a positive definite matrix which enables the application of elimination technique in purely, sequential process without pivoting especially for a banded stiffness matrix. The whole process is applied within the computer program. Once, the unknown vector of joint displacements is established, the internal member-end forces can be obtained by back-substitution of individual stiffness matrix equation of members.

3. COMPUTER PROGRAM

A Computer program is developed and written by FORTRAN-77 to handle the mathematical formulations, numerical solution and calculations of joint displacements, soil pressure, and member-end tractions. The main segment reads the input data, assembles the load vector and prints the output results while a group of subroutines are used to perform the stiffness matrices of structural elements, wall panels, raft foundation and soil; solve for unknowns joint displacements; calculate the member-end tractions. The input data are the material properties, joint loads, x- and y-coordinates of the joints, dimensions of various elements, joint numbers, and control parameters. The output results are the soil pressure at the contact interface, joint displacements, and member-end forces,

4. APPLICATION

A 12-storey framed building as shown in Fig.1 is chosen to demonstrate the method of analysis. The columns have a cross section of 0.5 x 0.5m for the first two levels. The section decreases by 0.05m every two levels. All beams have cross section of 0.25 x 0.60m. The raft thickness is 0.7m. The thickness of concrete wall panel is 0.25m. Loads and properties: Dead Load $= 15kN/m'$ + dead weight of panels; Live Load $= 12kN/m'$; Lateral Load $= 100kN/floor$; Ec $= 25 \times 10^6 kN/m^2$; $\mu_c = 0.17$; Es$= 5 \times 10^4 kN/m^2$; $\mu_s = 0.40$

Six cases of wall panel configurations as shown in Fig.2 has been considered. Only one parameter is selected for comparison among these cases and that is the horizontal floor displacements as shown in Table 1.

5. CONCLUSIONS

The two-dimensional analysis of framed buildings having arbitrary walls as structural elements has been presented which simulates the actual behaviour of the structure. Other conclusions may be drawn from this study:

a. Wall panels should be treated as structural elements increasing the stiffness of the structure upon which the floor drift is decreased.

Floor No.	Height m	Horizontal Floor Displacement for Various Cases, mm					
		1	2	3	4	5	6
1	4.5	0.29	0.22	0.13	0.11	0.11	0.08
2	7.5	0.93	0.41	0.20	0.16	0.15	0.42
3	10.5	1.61	0.64	0.27	0.23	0.23	0.73
4	13.5	2.17	0.55	0.28	0.29	0.30	1.00
5	16.5	2.25	0.30	0.29	0.38	0.45	1.17
6	19.5	0.76	0.23	0.33	0.52	0.67	1.42
7	22.5	2.00	1.76	1.86	0.83	1.11	2.22
8	25.5	6.54	6.20	6.28	1.21	1.67	3.27
9	28.5	11.60	11.10	11.20	1.93	2.53	3.88
10	31.5	16.50	15.90	15.90	5.89	3.01	4.93
11	34.5	21.60	20.90	20.90	10.40	3.73	5.76
12	37.5	26.60	25.80	25.90	15.80	4.72	7.10

Table 1. Horizontal Displacements at Floor Levels.

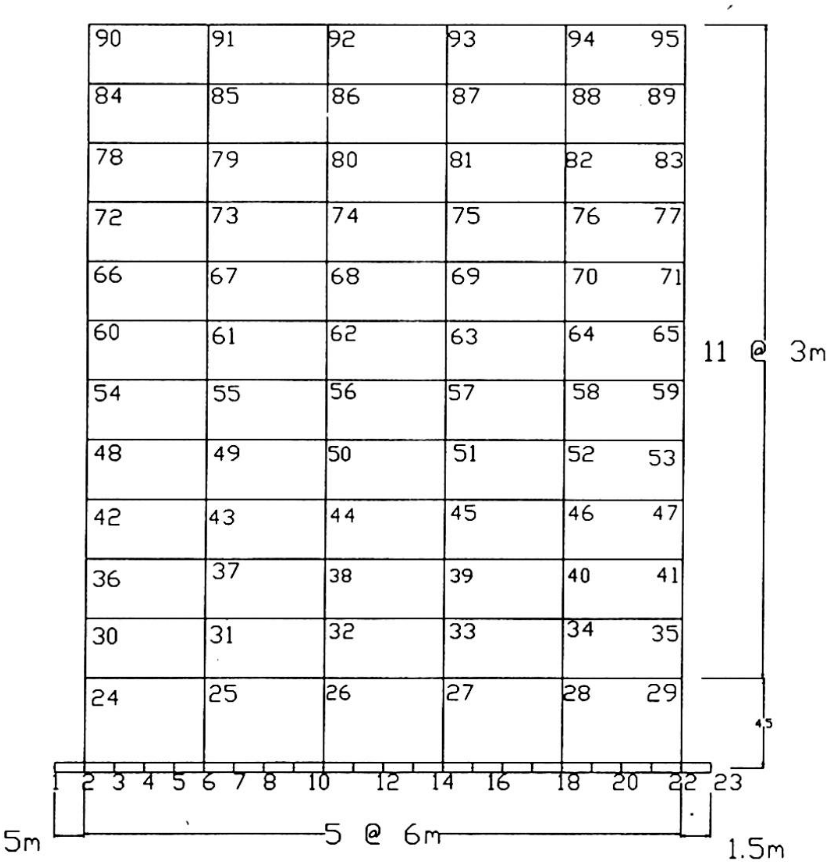


Fig.1. Geometry and Nodal Numbering

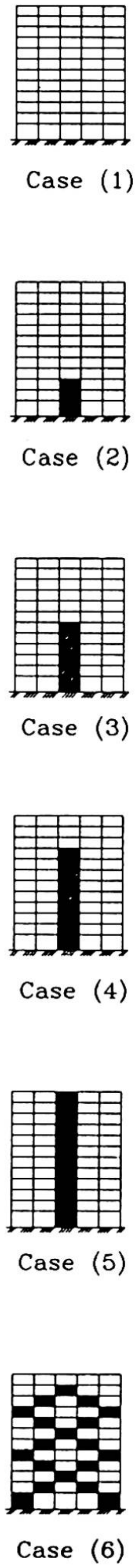


Fig.2. Panel Configurations

b. Wall panels forming a full shear wall is the best selection among all configurations. The shorter the shear wall is formed, the effect in enhancing the stiffness is reduced as can be observed in cases 3, 4, and 5.

c. Configuration of case 6 may be considered as shear wall equivalent, but on the account of using nearly twice the number of wall panels.

d. The presented method of analysis can be applied to concrete as well as to steel structures with concrete or metallic sandwich panels.

REFERENCES

1. GHANEM Z., EL-KAFRAWY A., and NASSEF M., Effect of Infill and Openings in Shear Walls on the Lateral Resistance of Tall Buildings. Proceedings of the Second Arab Conference on Structural Engineering, pp.23.1 - 23.21. University of Jordan, 19-22 April 1987.

2. GOVINDAN P., LAKSHMIPATHY M., and SANTHAKUMAR A., Ductility of Infilled Frames. ACI Journal, Vol.83, No.4, July-August 1986, pp.567-576.

3. SOBAIH M. and ABDIN M., Seismic Analysis of Infilled Reinforced Concrete Frames. Journal of Computers and Structures, Vol. 30, No. 3, 1988, pp.457-464.

4. KOST G., WEAVER W., and BARBER R., Nonlinear Dynamic Analysis of Frames with Filler Panels. Journal of Structural Division, ASCE, Vol.100, No.ST4, April 1974, pp.743-757.

5. UMARY J., Behavior of Framed Structures Having Wall Panels. M.Sc. Thesis, Jordan University of Science and Technology, Irbid, Jordan, June 1990.

6. LIAUW T., Tests on Multistorey Infilled Frames Subject to Dynamic Lateral Loading. ACI Journal, Vol.76, No.4, April 1979, pp.551-563.

7. GOYAL B., SHARMA S., MITTAL M., and AHMAD J., Need for a Rational Design of Multistoreyed Buildings with Shear Walls: a Case Study. The Indian Concrete Journal, Vol.59, No.4, April 1985, pp.107-110.

8. RESHEIDAT M., Soil-Foundation-Structure Interaction of Framed Structures. 8WCEE Proceedings, Prentice-Hall Inc., New Jersey, 1984, Vol.III, pp.777-784.

9. RESHEIDAT M., The Complete Analysis of 2-D Framed Structures. Proceedings of the First National Conference on Science and Technology for Buildings, 4-14 December 1984, University of Khartoum, Sudan.

10. WEAVER W. and GERE J., Matrix Analysis of Framed Structures. 2nd Edition, D. Van Nostrand Reinheld Company, New York, 1980.

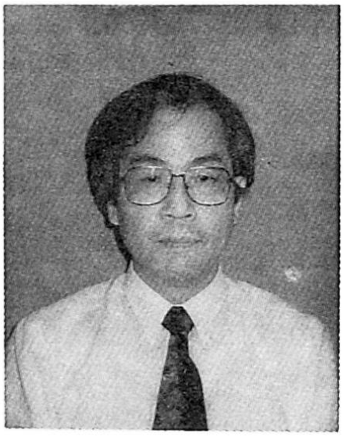
11. WEAVER W. and JONSTON P., Finite Elements for Structural Analysis. 1st Edition, Prentice-Hall Inc., New Jersey, 1984.

New Construction System of High Rise RC Buildings

Nouveau système de construction de gratte-ciel en béton armé

Neues Methode zur Errichtung von Stahlbeton-Hochhäusern

Hiroo TAKADA
Research Engineer
Technol Inst. of Shimizu-Corp.
Tokyo Japan



Hiroo TAKADA, born 1942, obtained his doctor of engineering at the Tokyo Science University. He has devoted the last 23 years to the study of new construction methods.

SUMMARY
The current labour shortages on sites can provide a good opportunity to reform the construction industry's labor-intensive nature. This could be accomplished by focusing on technological advances for construction production processes, thus completely reforming and modernizing the construction industry. The authors have developed the new concept of high rise building construction systems and it has been used to great effect at construction sites suffering from labour shortages.

RÉSUMÉ
Le manque de main d'oeuvre que nous connaissons actuellement sur les chantiers de construction peut être une excellente occasion pour réformer la nature gourmande en heures de travail de ce secteur d'activité. Pour atteindre ce but, il faudrait intégrer les progrès technologigues faits par les procédés de mise en oeuvre, opérant ainsi une réforme et une modernisation complète de la construction. industrielle. Les auteurs ont développé un nouveau systéme de construction de gratte-ciel, qui a été utilisé de manière très efficace sur les chantiers souffrant d'un manque de main d'oeuvre.

ZUSAMMENFASSUNG
Der gegenwärtige Arbeitskräftemangel auf der Baustelle kann als guter Anlaß zu einer Umstrukturierung der bisher arbeitsintensiven Arbeitsweise im Bauwesen dienen. Eine solche Umstrukturierung oder Modernisierung wäre zu erreichen durch die Einbeziehung technologischer Neuerungen in den Produktionsprozeß auf der Baustelle. In diesem Sinne haben die Verfasser ein neues Konstruktionssystem zur Anwendung bei der Errichtung von Hochhäusern entwickelt, das sich bereits auf verschiedenen von Arbeitskräftemangel geplagten Baustellen hervorragend bewährt hat.

1. INTRODUCTION

For the past few years, construction industry has been benefiting from the steady growth of other industries in Japan, and a number of high-rise RC buildings are currently being carried out or planned. However, looking at the situation that existed in many construction sites, it can be said that the construction industry is struggling to keep up with the rapidly increasing number of orders it is receiving. The main reason for this struggle is the shortage of labor that is a result of the labor-intensive nature of construction work. Although this is an old problem in the construction industry, no solution has ever been found.
The authers are of the opinion that construction site labor shortages can provide a good opportunity to reform the industry's labor-intensive nature. It has been proposed that the current shortage of labor at construction sites is not just a transitory problem but a way to help the construction industry veer away from labor-intensiveness. This could be accomplished by focusing on technological advances for the construction production processes, thus completely reforming and modernizing the construction industry.

2. THE CONCEPT OF THE INTEGRATED CONSTRUCTION SYSTEM

We have developed a new concept of high rise building construction system and named it the concept of integrated construction system. It has been used to great effect at construction sites suffering from labor shortages.
Fig.1 illustrates the basic concept about the concept of the integrated construction system. The construction work method eventually adopted at the construction site should allow for the best balance posible among quality, the construction term, economy, and safety. This sort of study is carried out in conventional and industrialized methods. However, regarding the concept of integrated construction system, subsystems for the construction of each section are selected from among the available methods, regardless of standard conventional and industrialized procedures. Thus, a greater range of selection is possible.
The advantage of the conventional methods is that all contructors, construction planners and managers, and design supervisors engaged in construction have shared a common tradition and experience with them ever since concrete developed. Structually, the conventional methods have no trouble with joints, because there are few jointings of successive concrete pour within the form-work, and homogeneous

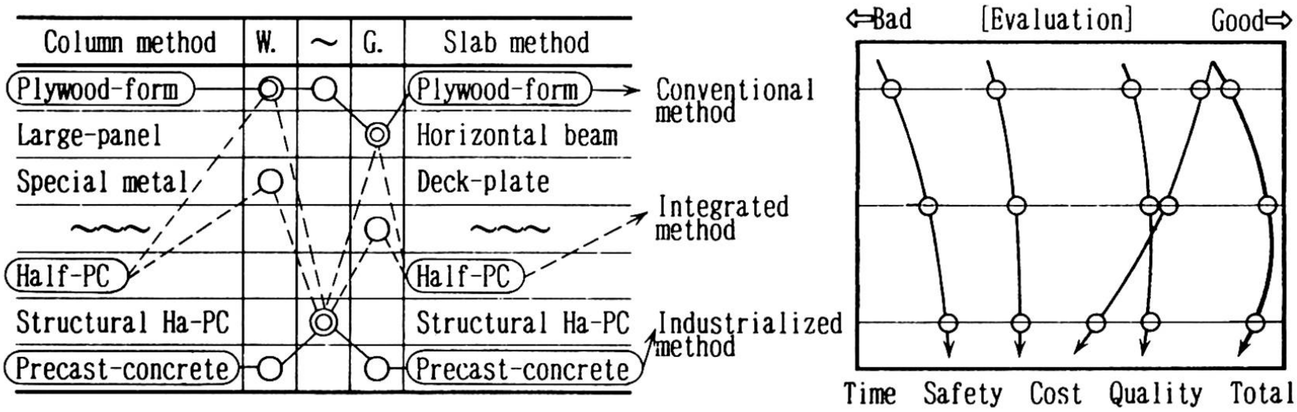


Fig. 1 The basic concept of the integrated construction system

concrete structure can be produced, and the generation of high-level local stress can be avoided, From the standpoint of concrete quality, superior construction is possible with good shielding performance against water, heat, fire, and noise. In production, the conventional methods have many useful properties, including good compliance and constructivity. The conventional methods provide more problem-free interfaces than industrialized methods do, making flexible production possible and no need for subsequent processes such as the treatment of joints. Also, the conventional methods are better in price, although it's said that in quality and speed of construction they are inferior to the industrialized methods.
With regard to precast concrete member mutual interfaces, the industrialized construction methods is based on the idea of "strictry conventions" that are simplified and standardized so as to be in common only within an individual construction methods. This makes it possible to have a closed system and is an attempt to achieve prefab effectiveness through mass production. Thus the industrialized methods have superior performance in term of quality and construction speed. But in term of price it is said to be inferior to the conventional methods. The integrated construction system adopts the open system, in which they are as "simple, flexible agreements" as possible, to make it possible to adopt a variety of methods and members for every position. Thus its goal is not to achieve the prefab effect through mass production as in the industrialized methods, but rather to achieve an organic-effect for construction work as a whole by using simple, flexible agreements to cleverly incorporate semi-finished prefabricated materials into construction plans, for the purpose of relieving the labor situation and shortening construction term.

3. APPLICATIONS

3.1 APPLICATION EXAMPLE

The concept of the integrated construction system has been applied to many high-rise buildings. The schedule of building construction and the site planning using the concept of the integrated construction system are shown in Fig.2 and Fig.3.

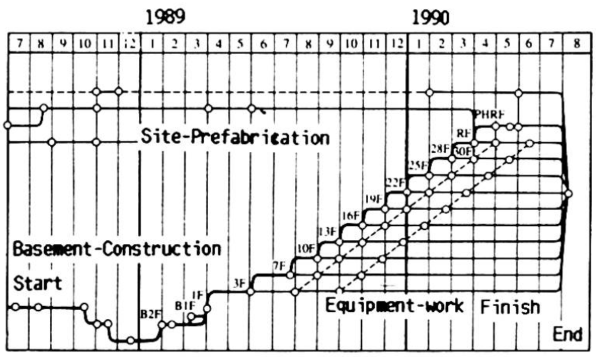


Fig. 2 The schedule of building construction

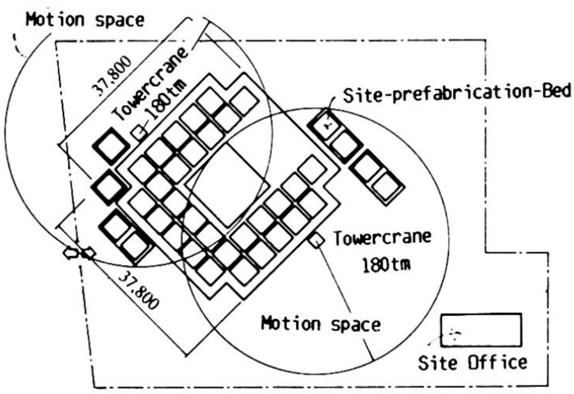


Fig. 3 The site planning

This building is designed for the construction of a multiple-dwelling house 30 stories of reinforced concrete structure. The blocking construct area method as shown in Fig.4 and the separated horizontal-and-vertical construction method is adopted.

3.2 ONE ACTUAL EXAMPLE

The basic plan of "SST" is shown in Fig.5. The case of SST is given as one actual example of the concept of the integrated construction system and it has been developed for the construction of super high-rise RC multi-dwelling buildings. A construction planning of a high-quality and super high-rise RC apartment take such general conditions into consideration as a large volume of materials, long lifting distance, shortage of labor, job repetition, high-techniques required and facilities held. Then, the appropriate quantity of labor, materials, equipment, work term, etc. is decided for the whole system and the subsystems. Finally, SST that optimizes the entire body is created as shown in Fig.6. No particular methods is specified for each part of the building, but rather the conventiónal method, the half-precast concrete method and full-precast concrete method are all a vailable. The method for each part in selected, simulated, and determined from among them based on the size of the building, construction-terms, the building's location, and while seeking to optimize the construction work as a whole.

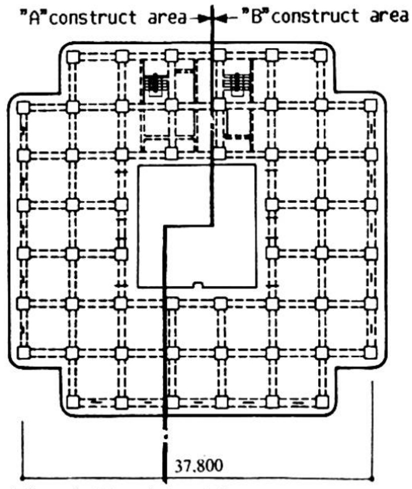


Fig. 4 The blocking area

Recently the shortage of skilled workers at construction sites has become a major contraint, and at many sites, SST is done at the pase of one foor every six to eight days under a plan using a specific formwork method for columns, a specialized formwork method or half precast concrete method for girders, the precast concrete method for walls, stairs and valconies and the large half precast concrete method for slabs.

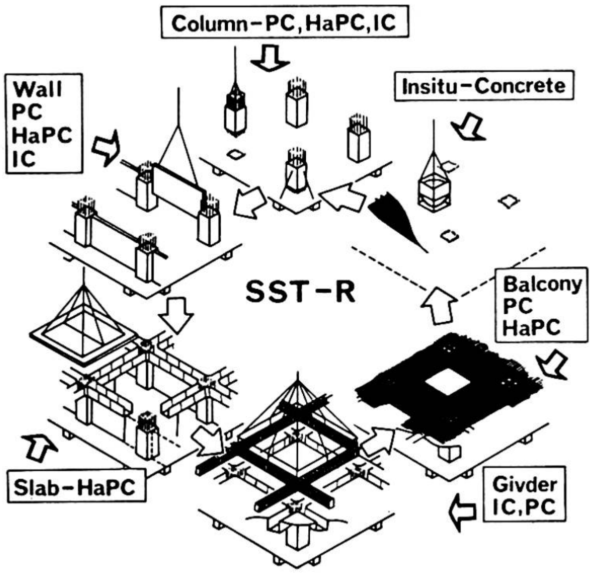


Fig. 5 The basic plan of SST

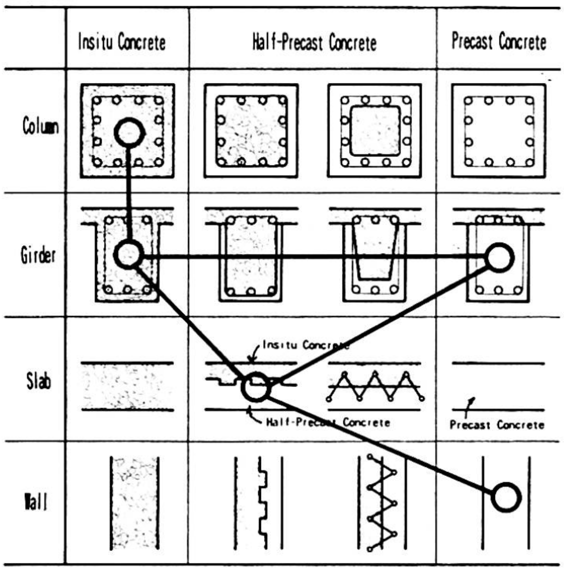


Fig. 6 The selected methods of SST for application example construction

3.3 WORK SCHEDULING

When one cycle of detailed work is planned to improve the operation rate of workers, cranes, forming materials, temporary facilities, etc, the work scheduling is effective to plan repetitive work. Work repetition brings improves learning

and productivity, stabilizes quality and favorrably influences work safty.
In order to realize a high production on the high-rise building construction, not only repetitive work but also a fixed members of workers and standardizing crew size is required. The multi-activity-chart method is applied to work scheduling of application example construction in which seven work teams repeat their work at the same cycle in two blocks simultaneously, as shown Table 1. Multi-activity-chart is a timetable which indicates each work team's schedule, and who, when, where, and what they do. In this table, the holizontal axis shows each work teams and the vertical axis shows workdays.

Team	Crane		Carpenter	R-Bar Placer		Mecanician	PC-Labor
Number / Days	1	1	9	5	3	2	4
1 st	Scaffolds SD.ALW Delivering	PC-Girders Setting PC-Slabs Setting	Columns Form Erection Girders Panels Stripping	Columns Re-Bar Pre-fab.	Girders Re-Bar Pre-fab.	Columns Re-Bar Jointting Girders Re-Bar Jointting	Half-Slabs Pre-fab.
2 nd	Columns Concrete Pouring	Columns Re-Bar Erection PC-Balcony PCCorridor Shoring	Girders Panels Erection				
3 rd	PC-Slabs Setting Girders Re-Bar Erection	Slabs Re-Bar Erection	Slabs Supporting	Columns Re-Bar Erection		Inspection	
4 th	PC-Walls Setting PC-Stairs Setting	Slabs Concrete Pouring	Columns Form Stripping Columns Form Reforming	Slabs Re-Bar Erection Girders Re-Bar Erection	Girders PC Re-Bar Erection	Columns Re-Bar Jointting	

Table 1. The multi-activity-chart method applied to work scheduling of application example construction

4. CONCLUSION

The concept of the integrated construction system is now popular in Japan and SST has been applied to many high-rise buildings in a relatively short time. The concept of the integrated construction system makes it possible to complete construction work satisfactorily and successfuly in shorter periods and thereby to save manpower and conserve materials. The effects of the concept of the integrated construction system on the economy of the construction work were examined on site as follows:

1)The labor productivity ratio of the integrated construction system to the conventional method was 2 to 1.

The comparison of the amount of labor beween the actual amount of labor invested on site in which the integrated construction system was used, and the trial-calculated amount of labor planed to be invested using conventional methods is shown in Fig.7.

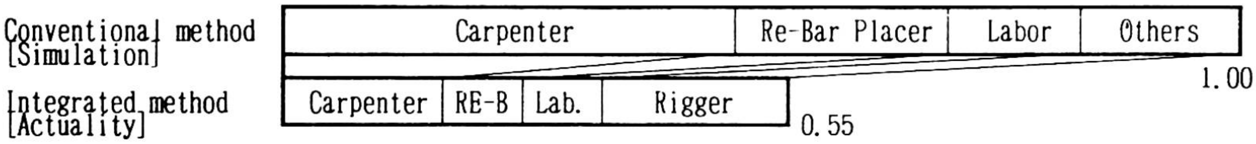


Fig. 7 The comparison of the amount of labor

This diagram shows only the quantity of labor expended, but it is clear that its quality is reduced along with its quantity, as fewer skilled workers and more unskilled workers are employed.

2)The construction period required for the integrated construction system was approximately half as long as that for the conventional method.

The comparison of the construction speed beween the actual amount of the construction speed on site in which the integrated construction system was used, and the trial-calculated amount of construction speed planed to be invested using conventional methods is shown in Fig.8.

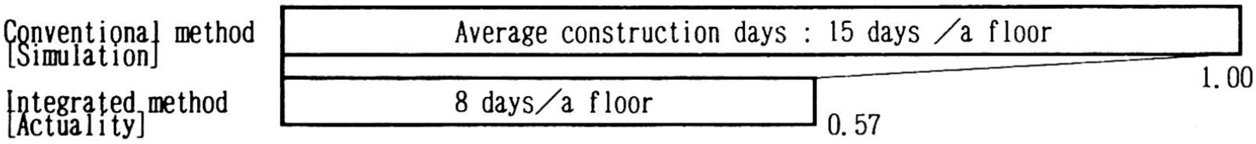


Fig. 8 The comparison of the construction speed

3)No different expenses are seen between the integrated construction system and the conventional method.

The comparison of the construction expenses beween the actual value of the construction expenses on site in which the integrated construction system was used, and the trial-calculated value of the construction expenses planed to be invested using conventional methods is shown in Fig.9.

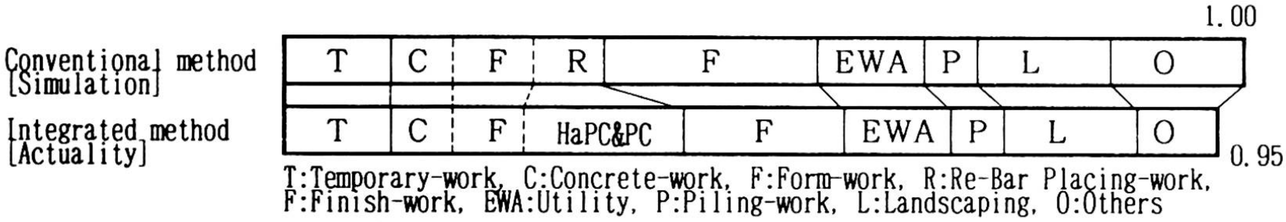


Fig. 9 The comparison of the construction expenses

4)The integrated construction system can be of great use for labor saving and construction term shortening of high-rise buildings.

The rate of labor decrease at every construction site in which the integrated construction system was used is shown in Fig.10.

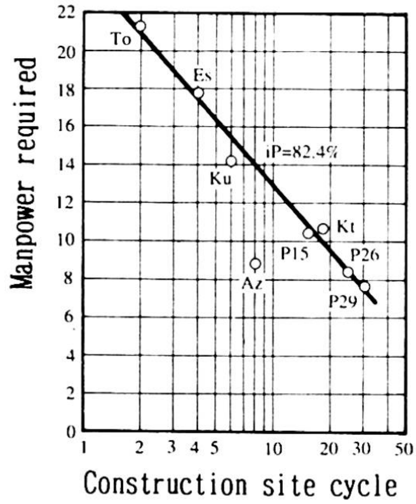


Fig. 10 The rate of labor decrease

REFERENCES

1.H.TAKADA, S.KOKUSYO, Y.MATSUZAKI, K.KOBAYASHI, New Composite Slab System by Large Scale PC Panel Cast in Site. IABSE 13th CONGRESS June 6 ～10,1988 pp957～pp962

Scope Towers at New Delhi, India

Tours Scope à New Delhi, Inde

Scope-Türme bei Neu Delhi, Indien

S. RANGARAJAN
Principal Consult.
STUP Consult. Ltd.
Bombay, India

A.S. PATKIE
Principal Consult.
STUP Consult. Ltd.
Bombay, India

Rajiv AHUJA
Senior Consultant
STUP Consult. Ltd.
Bombay, India

S. Rangarajan, born 1943, obtained his Master's degree in Civil Engineering from Madras. After four years in teaching, he joined STUP Consultants Ltd. Involved in the design of major concrete structures, prestressed concrete bridges, flyovers and highrise buildings.

A.S. Patkie, born 1947, graduated in Civil Engineering from Bombay University and obtained Masters Degree in Management Studies from Bombay. At STUP Consultants Ltd. Involved in the technical and financial planning as well as project management of major projects in civil engineering.

Rajiv Ahuja, born 1955, obtained his Master's Degree in Civil Engineering from Delhi. After being involved in geotechnical investigation for various Civil Engineering structures for three years, he joined STUP Consultants Limited. Involved in the design of multi-storeyed buildings and bridges.

SUMMARY
This twenty two storeyed, high rise twin tower structure, nearing completion at Laxmi District Centre, Delhi, is a modern office complex for SCOPE. This imposing structure demonstrates the versatile utility of structural concrete in its various forms. Some of the salient features of construction include use of large diameter bored piles for foundations, large cast-in-situ reinforced concrete construction, raft, slipformed lift cores, precast flooring and pumping of concrete to various locations of this 110 m diameter building.

RÉSUMÉ
Il s'agit d'une tour jumelée de vingt-deux étages en voie d'achèvement à Laxmi, quartier central de Delhi, représentant un complexe de bureaux ultramodernes. Cet ouvrage imposant fournit la preuve de la diversité d'utilisation du béton structural sous ses formes les plus variées. L'article expose quelques unes des caractéristiques essentielles de cette construction, à savoir l'exécution de pieux forés à grand diamètre pour les fondations, le radier en béton armé coulé sur place, les cages d'ascenseurs réalisées en coffrage glissant, la préfabrication des planchers et le pompage du béton aux différents niveaux de ce bâtiment de 110 m de diamètre.

ZUSAMMENFASSUNG
Die sogenannten Scope-Türme, ein zweiundzwanzig Geschosse umfassender Bürokomplex im Laxmi-Distrikt-Zentrum, stehen kurz vor der Vollendung. Dieser eindrucksvolle Doppelturm führt den vielseitigen Nutzen von Konstruktionsbeton vor Augen: Grossbohrpfähle unter einem ausgedehnten Ortbetonträgerrost dienen als Fundament für die in Gleitbauweise hochgezogenen Liftschäfte. Neben einem Fertigteildeckensystem gelangte in dem grossen Gebäude von 110 Durchmesser auch Pumpbeton zum Einsatz.

1. GENERAL

The twin tower office complex for the Standing Conference of Public Enterprises, SCOPE, is nearing completion as a major facility of the district centre and shall house a number of public enterprises, including the Oil and Natural Gas Commission, ONGC.

The structure consists of two high rise curvilinear tower blocks, rising above a four storied circular podium block which includes two basements for car parking and a central mushroom. The podium flares outwards above the first floor level. The two towers rise above the circular podium and are of different heights, one having twenty two storeys and the other, seventeen. There is a provision for a heliport over the terrace. The 110 m diameter of the base of the structure gives an idea of its imposing size. The four storeyed circular podium block encompassing both the towers, flares outwards above the first floor level. With a total built up area of over 100,000 Sq.m. SCOPE Towers is to become a landmark on Delhi's skyline. (Ref. Fig.1, Fig.2, Fig.3).

2. STRUCTURAL ASPECTS

2.1 Analysis and Design

The tower structures which rise from the sprawling podium block are isolated from the low level structure through expansion joints. The method of analysis and detailing to arrive at the framing plans was finalised after considering a number of basic structural decisions affecting design and construction. These included precasting of typical floors above podium level to reduce elaborate in-situ staging and shuttering for slabs and to reduce the time cycle for the multistoreyed portion.

Installation of lifts takes considerable time and hence it was decided to slipform the four lift cores, so that the lift core and machine room could be in position much ahead of the other floors. Additionally, the

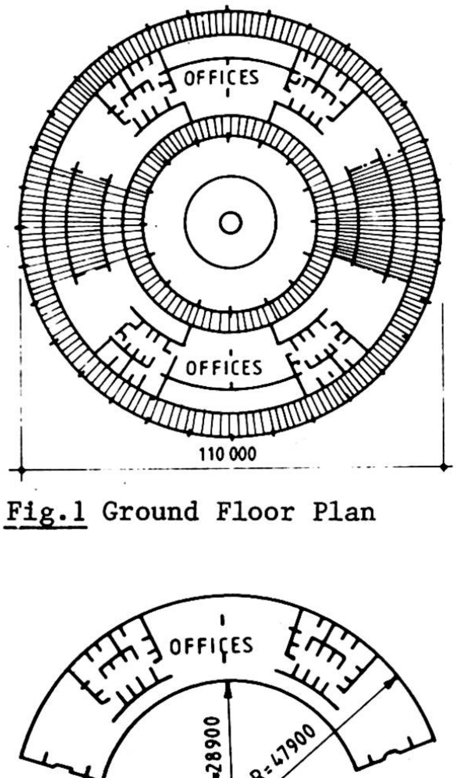


Fig.1 Ground Floor Plan

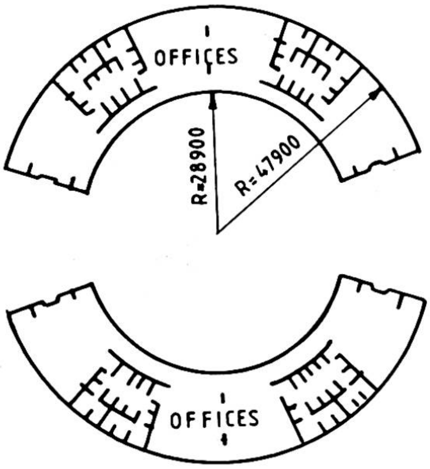


Fig.2 A typical floor plan

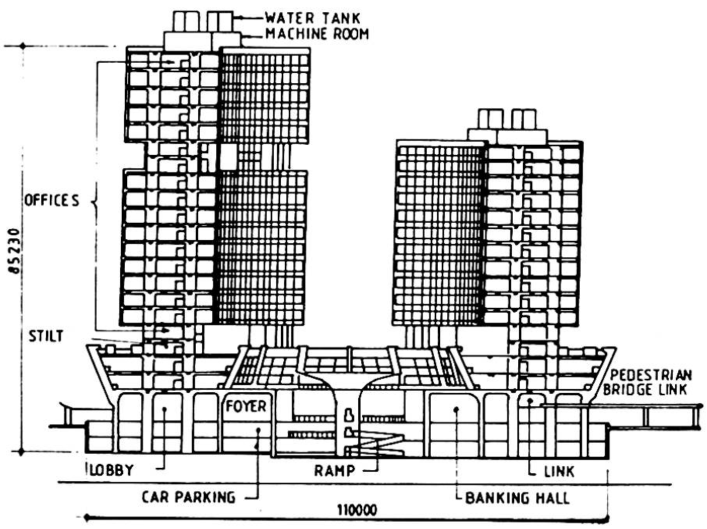


Fig.3 Sectional Elevation

machine room roof could be used for installing the cranes used for lifting precast elements and other construction materials. Expansion joints, construction joints and the sequence of construction especially to the large raft were also finalised.

The structural analysis for dead, live, wind and seismic loads was carried out by a three dimensional frame analysis, for the tower structure. The building was modelled as a space frame with degrees of freedom reduced by imposing the condition that the floor slabs were rigid in their own planes. The lift walls were considered as coupled shear walls. The program used was based on standard stiffness matrix method of frame analysis. Due to repetitive geometry of the building, the program generated data base on details of a typical floor. In addition to generation of element node relationship, co-ordinates, member types and sectional properties, the loading data was also generated from basic input information.

The structural analysis was carried out in two parts. The first part dealt with the dynamic analysis involving the computation of frequencies, mode shapes and earthquake forces on the basis of mode superposition method as per the recommendations of the Indian Standard IS:1893, criteria for Earthquake Resistant Design of Structures. Fig.4 shows the lumping of masses and Fig.5 the mode shapes. The second part deals with the analysis of the structure for dead and live loads in combination with seismic and wind loads, the latter being obtained from wind tunnel tests.

2.2 Wind Studies

To evaluate the wind pressure distribution on the two curved towers, wind tunnel testing was carried out during December 1985 at the Indian Institute of Technology, Delhi. This helped the scientific evaluation of the wind effects on the structure with the given shape, orientation, tower heights and relative disposition of the two tower blocks. The wind tunnel test was carried out on the structural model mounted on a turn table, in different orientations with respect to the direction of the wind. Contours of wind pressures over the surface of the structure were furnished by the Indian Institute of Technology, Delhi, for different directions of wind. Based on these data, wind pressures over the width and height of structure were evaluated and the wind loading considered suitably in the space frame analysis. Fig. 6 shows typical wind pressure contours on the structure. The wind pressure diagram adopted for analysis is shown in Fig. 7.

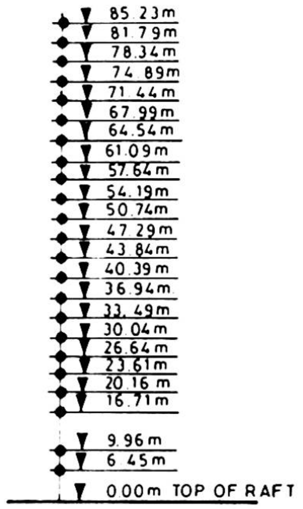


Fig.4 Mass lumping at floor levels for Dynamic Analysis

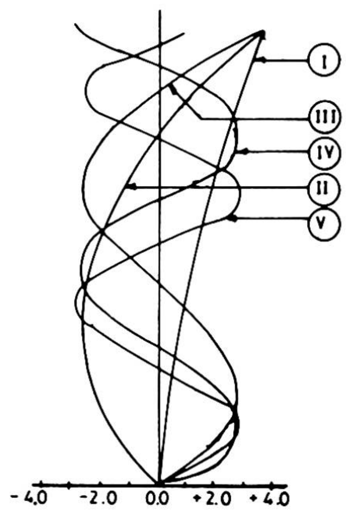


Fig.5 Mode Shape Diagram

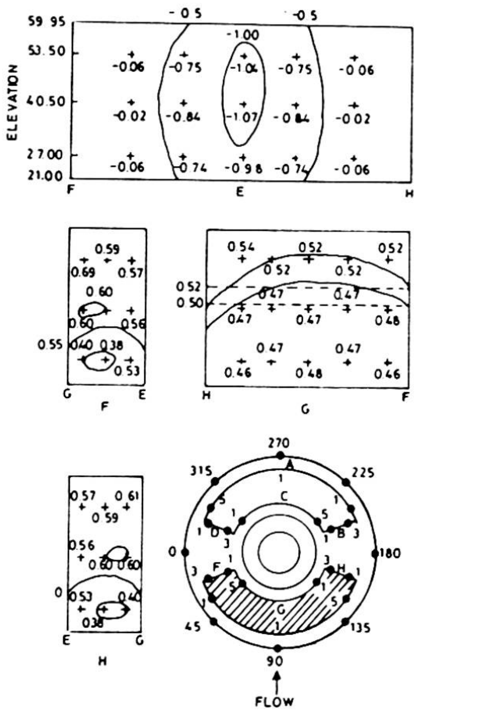


Fig.6 Contours of Pressure Coeffs. for 90° Orientation of Wind (Block II)

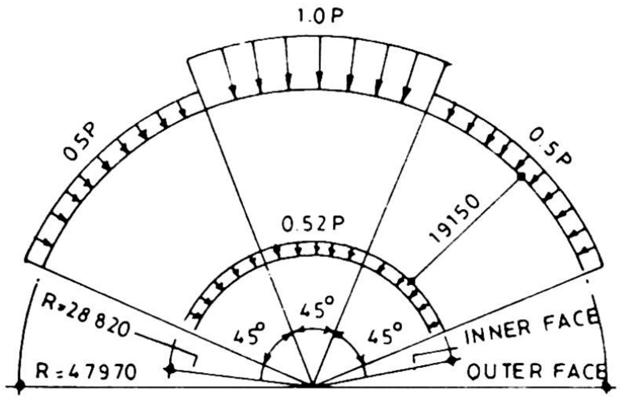


Fig.7 Wind Pressure Distribution Plan

While the global space frame analysis results were useful for evaluating moment, shear and torsion in the main framing elements for the various loading cases, under their various combinations with load factors; local grid and space frame analysis were carried out for determining forces in secondary members. The structural design of the members was carried out by the limit state design.

2.3 Foundations

The sub-soil at site is alluvial in nature upto 30 m below ground level. It was decided to adopt large diameter cast-in-situ bored piles going upto a depth of 20 m below the basement raft which is located at about 11 m below ground level. The pile diameters chosen were 1200mm, 700mm and 600mm. Though the maximum safe load capacity of 1200mm pile is 5500 kN, it was tested for a load of 14000 kN. The ground water table at site could rise upto 1.0 m below the ground level and hence the basement is subjected to a high water pressure. It was, therefore, necessary to provide piles for the entire base covering the towers and the low level structures. Fig.8 shows a typical arrangement of piles below a lift core. The piles are capped by a common circular base raft of about 110 m diameter, with thickness ranging from 0.9 m to 1.5 m. A tapered retaining wall with maximum thickness of 600 mm is provided all round the basement.

2.4 Superstructure

The structure is curved in plan and is framed by curved beams with large spans upto 180 m, heavy columns, lift cores and end shear walls. Each tower block comprises of two lift cores which are slipformed. During construction, two halves of each lift core were coupled by steel beams which were embedded later in a cast-in-situ 300mm thick diaphragm slab connecting both the halves. Floors upto mezzanine level are cast-in-situ. Above this level, precast elements are used and they are simply supported between circumferential cast-in-situ beams. The maximum length and weight of these elements are of the order of 7.0 m and 20 kN, respectively. A total of 18 types of elements of various sizes are used. Fig. 9 gives details of structural framing in a typical floor and Fig.10 shows the type of precast element used.

Another attractive feature of this office complex is a central mushroom shown in Fig.3. It is a centrally located structure with a 5.3 m diameter vertical shaft with an overall height of 19.0 m. It is topped by a doubly curved shaft of 100 mm thickness with a top diameter of 23.0 m. A three-dimensional analysis for the shaft was carried out by using SAP IV PROGRAM.

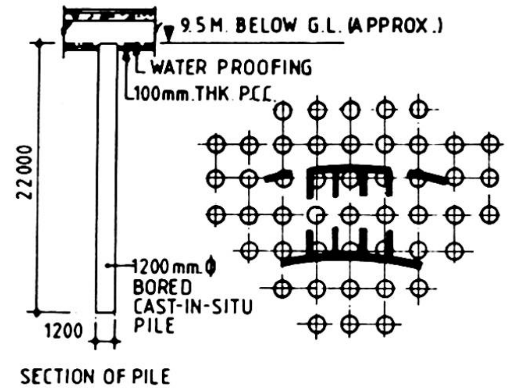


Fig.8 Typical arrangement of piles below lift core

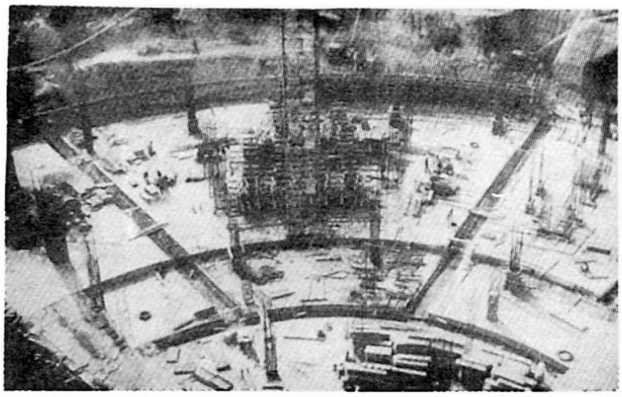


Fig.9 Base Raft with shrinkage gaps

3. CONSTRUCTION FEATURES

3.1 General

Even during the finalisation of structural framing, the following basic decisions emerged.

3.1.1 Precasting of floors was to be adopted to the maximum extent possible above the ground level. This would help in avoiding extensive in-situ staging and shuttering, and result in better quality control of the elements, and a faster rate of vertical construction.

3.1.2 The lift cores were slipformed in order to have the following facilities.

- Generally, the erection of lifts takes considerable time and becomes critical in the schedule. Hence, if the lift core is available ahead of rest of the structure, installation of lifts could begin in advance.

- The roof of lift cores could be used for installing cranes for lifting and installing precast elements and other lifting requirements.

3.1.3 A major construction consideration was the casting of the base raft with a thickness of 1.5 m under the tower portion and 0.9 m outside. Due to the pressure of the high water table it was decided that the 110 m diameter base of the structure will not have any expansion joints. However, circumferential and radial shrinkage gaps were provided at predetermined locations to facilitate casting of raft in segments and also to enable concrete to undergo shrinkage in smaller parts. The shrinkage gaps were concreted 28 days after adjacent parts of raft were cast. This helped in greatly reducing residual shrinkage strains in the concrete in the monolithic raft. The base of the raft and the sides of the basement retaining walls were waterproofed to withstand ground water pressure.

3.2 Concrete Pumping

Another interesting aspect of construction adopted by the contractors M/s. Larsen and Toubro was the use of pumped concrete in conjunction with a batching plant, for efficient production and delivery of concrete for this sprawling structure requiring large quantities of concrete at various locations and different heights. Pumped concrete was used for casting of the huge base raft (concrete quantity 13,000 Cu.m.) from lower basement floor upto a height of 50.0 m. To pump the concrete efficiently upto desired heights, additional

Fig.10 Slipformed Lift Cores

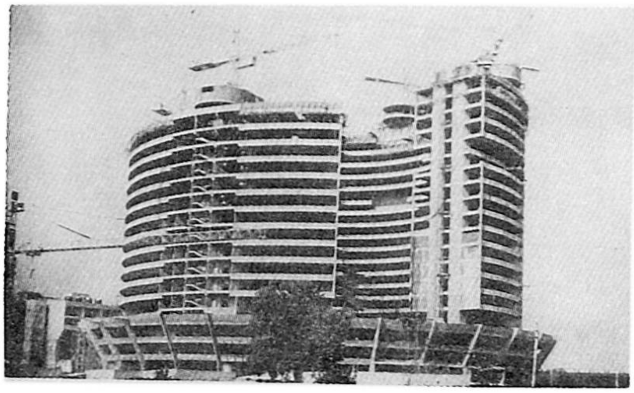


Fig.11 Building under Construction

pumps at intermediate levels were installed. To avoid choking of pipes and for transporting concrete without affecting the workability, higher slumps were used ranging from 75 mm to 110 mm. Superplasticizers were added to achieve the desired slumps without appreciably increasing the water cement ratio Concreting at higher elevations is proposed to be done with the help of buckets which will be lowered by four cranes mounted on top of the lift cores.

3.3 Slipforming

Slipforming of lift cores was a highly skilled job which required careful and proper planning, in view of the complex curved shape of the lift cores. Both the halves of the lift core were slipformed together and openings were left wherever necessary by assembling and inserting a wooden frame while slipforming. A check list was prepared to include complete details of reinforcement, pockets, openings and other embedments. The large size working platform was operated using a system of centrally controlled hydraulic jacks. Verticality of lift core was regularly checked by means of water level. Water level indicators were installed at various locations on slipforming platform to check any tilting. Concrete was taken upto the working platform by a bucket which moved inside the lift core and was operated by winches installed at the base raft level. Access to the platform was provided through a moving trolley. The normal rate of slipforming was 100 to 150 mm per hour. The lift core of about 85 m. height was complete in about a month's time.

In view of the decision to slipform the core, it was necessary to work out details of embedments in the walls of lift core, as these were to be kept for subsequent connection of members joining the lift core, perpendicular to the plane of slipforming. These embedments had to be fabricated before commencement of slipforming, kept in position precisely and securely and then concreting had to be done effectively around such embedments to avoid honeycombing.

3.4 Material Quantities

For a total built up area of about 100,000 Sq.m. the quantities of major construction materials are as follows:

Concrete of various grade	:	50,000 Cu.m.
High yield strength bar reinforcement	:	7,500 t
Number of piles: 1200 mm diameter	:	272 Nos.
700 mm diameter	:	109 Nos.
600 mm diameter	:	77 Nos.
Number of precast elements	:	6,000 approximately.