AMERICAN INSTITUTE OF STEEL CONSTRUCTION
The Wrigley Building, 400 North Michigan Avenue
Chicago, IL 60611

MODERN STEEL CONSTRUCTION

NUMBER 2 • 1987

THIS ISSUE

Efficiency and Elegance—with Steel
A Landmark Stands Tall
A Study in Gothic—and Steel
Advances in Bridge Construction
Chicago's Newest Waterfront

DECK DESIGN DATA SHEET

NO. 9

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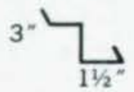

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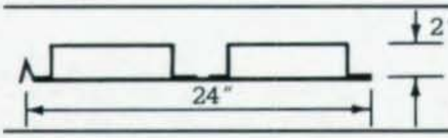

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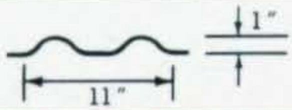

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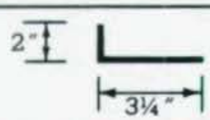

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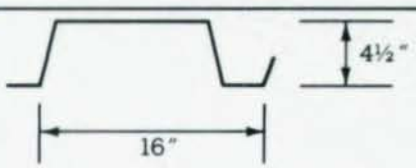

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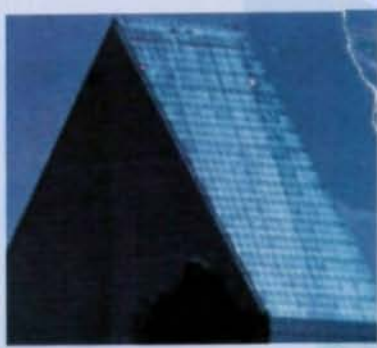

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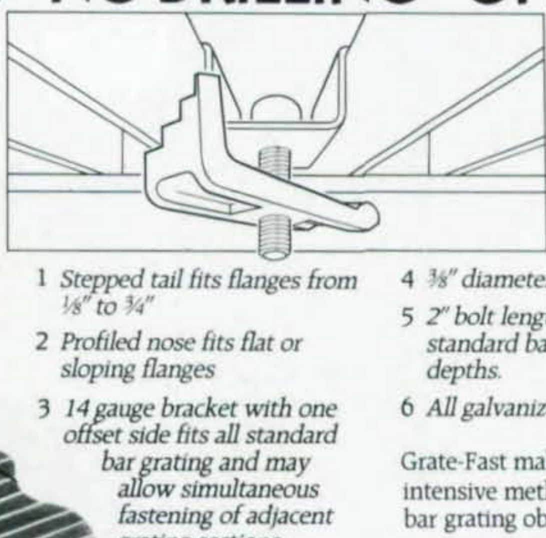

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ONE LIBERTY PLACE

Efficiency and Elegance in the Cradle of History

by Charles H. Thornton, Udom Hungspruke, Thomas Z. Scarangello and Scott Pratt

As One Liberty Place rises, the golden age of the skyscraper era of the 20s and 30s re-emerges on the Philadelphia skyline. Indeed, the building's facade borrows inspiration from the Art Deco influence of that period. Yet, despite similarities in form and appearance of such classics as the Chrysler Building in Manhattan, One Liberty Place can hardly be categorized as either an architectural or a structural twin.

The structure makes an elegant and optimistic statement about Philadelphia and its future. Consequently, it is a unique, dynamic design—influenced by the past but without duplication of all the forms, materials or structural systems. A building of this size designed in the 20s or 30s would have had between 40 to 50 lbs. of structural steel per square foot; not 23 psf as constructed.

Streamlined and powerful, the building's profile and structure represent a combination of boldness and refinement. It soars from bedrock along a slender shaft, utilizing a superdiagonal outrigger system culminating in a gracefully gabled top to do much to redefine the existing skyline.

Controversial from its start, One Liberty Place towers monumentally above Philadelphia's City Hall, previously the tallest building in Philadelphia and once (1880) the world's tallest building. A gentlemen's agreement between developers and the city, not backed by any law, had kept the top of all previous buildings below the elevation of William Penn's head on the statue atop City Hall. By proposing taller and more slender buildings which let more light and air into the city, city planners convinced the city these buildings would, in the long run, improve the quality of light and air at street level within the city.

One Liberty Place, at 945 ft, the ninth tallest building in the world, is sited on a prime downtown block on Market Street between 16th and 17th Streets. Office floors range from 24,000 sq. ft in the lower areas to 1,300 sq. ft at the peak. The 61-story tower contains over 1.3 million sq. ft of office and retail space.

Selection of Steel

Structural steel framing was chosen for its flexibility and high strength; and in particular, its ability to transmit large tensile and compressive forces efficiently and yet keep the size of members to a minimum. Built-up wide-flange sections were used for all outrigger diagonals and core and outrigger columns because of the large forces and required thickness of plates. Their use also facilitated fabrication and erection. (See Figs. 1 and 2.)

Floor System

The typical floor framing (Fig. 3) is composite W21 ASTM A-572 Gr. 50 steel beams spanning 44 ft from the building core to exterior face. As a result, the entire lease space within the tower is column-free. The structural slab is 3-in. composite decking with 2½-in. stone concrete topping. Floor beams were cambered to compensate for dead-load deflection under wet concrete placement.

Figure 1

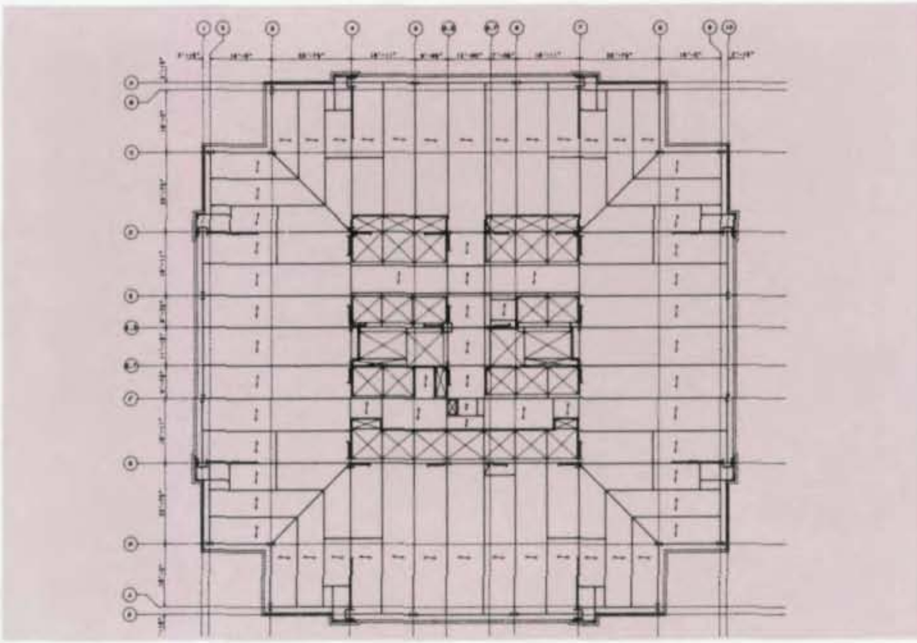


Figure 3. Typical floor framing plan

Figure 2

Structural Concepts Explored

In selecting the final structural system, several alternate lateral (wind) load resisting systems were evaluated fully by the design and construction management team:

- A tube system with column spaced 10 ft o.c. around the perimeter was studied. With this approach, columns approximately 40-in. wide and spandrels as deep as 72 in. were required. The steel weight with this approach was approximately 24 psf, excluding required transfer trusses at the base. The tube system was ruled out for the following reasons:
  a. Higher cost
  b. The developer wanted large windows with small columns along the perimeter and at corner offices.
  c. *The 72-in. deep spandrels required* an upset flange extending 30 in. above the floor slab which interfered with leaseability of space and accessibility to windows.
  d. With the parking, retail, entrance lobby and other architectural requirements at the base, many of the tube columns would have to be picked up or transferred above the ground floor. The added cost of these transfers totally prohibited the tube approach.
  e. The desire for corner offices resulted in the use of re-entrant corners, which structurally reduce the stiffness and, therefore, efficiency of the tube structure.
- *An exterior diagonally braced frame* was considered and deemed undesirable from an aesthetic and marketing point of view because of the multitude of diagonals interrupting windows. In addition, it was difficult to resolve the diagonal *bracing at the entrance lobby, retail lev*els and parking levels. The steel weight and cost of the diagonally braced frame scheme amounted to 22 psf of structural steel.

- A centrally braced core was studied, but determined uneconomical because of the extremely high steel weight caused by the slenderness of the core at a height-to-width ratio over 12. Drift and acceleration would have been extremely difficult, if not impossible, to control. In addition, large uplift forces on the foundation would have dictated extensive tie-down systems within the drilled pier foundation.
- A superdiagonal outrigger system which couples diagonal outriggers and exterior outrigger columns with a braced core (Figs. 1 and 2) proved the most economical (23 psf) structural system. This outrigger system met all the functional and architectural requirements of the project. It permitted small (W14 × 257) columns at all corners, eliminated the need for transfers at the base and provided almost unlimited flexibility at entrances and lower levels. It was the selected system.

Foundation Evaluation

After review of various foundation approaches, the design and construction team concluded that drilled caissons extended to and into rock would be the most economical foundation. This decision was reached after completion of borings and geotechnical studies by the site engineers established the close proximity of competent bedrock to the lowest building basement. Alternate studies using 1, 2, 3 and 4

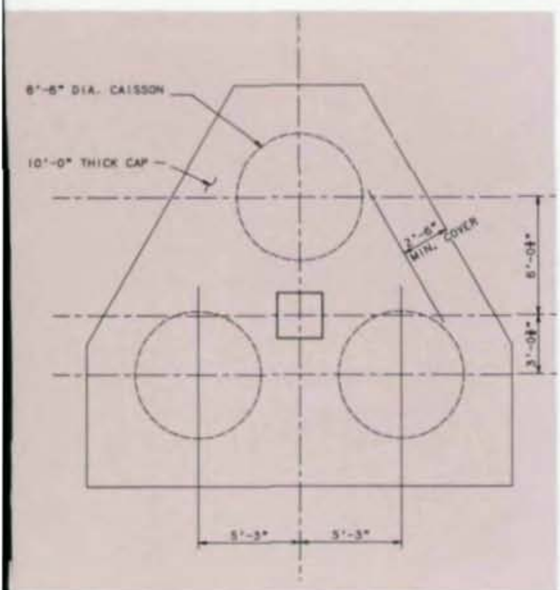


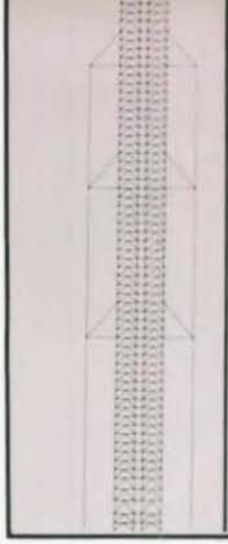
Fig. 4. Cluster caisson plan

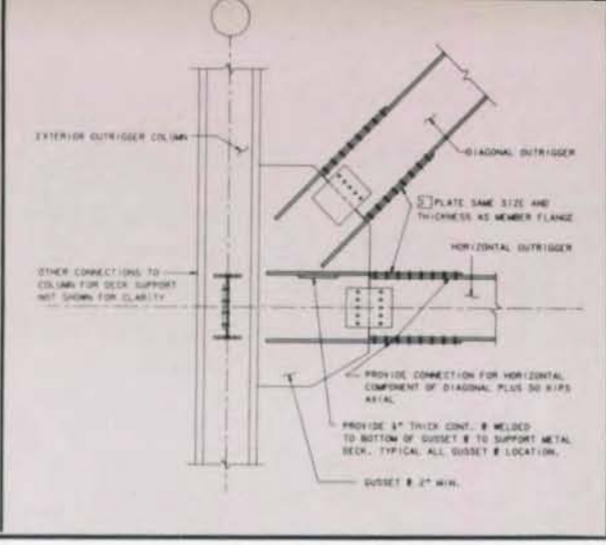
basements were undertaken. It was decided to use two basements, with drilled caissons to rock. To achieve an economical design, the team designed the caissons for a 4- to 8-tons per sq. ft (tsf) skin friction value in addition to 40 tsf of bearing. This enabled design of the length of rock sockets with a maximum allowable stress in the caisson shaft of 90 tsf.

To achieve further cost savings, clustered caissons were used under the heavily loaded corner core columns in lieu of using one large caisson (Fig. 4). The maximum total compressive load on these caissons was 17,340 kips. This approach eliminated the need for special oversized drilling equipment which was not readily available in the Philadelphia area. By maintaining a maximum caisson size of 8 ft in diameter, a more competitive bidding process and quicker foundation construction was possible, with reduced overall foundation cost. The foundations were completed in six weeks.

Lateral Load Resisting System

The selected lateral load resisting system is a superdiagonal outrigger scheme comprised of a 70-ft × 70-ft braced core coup-





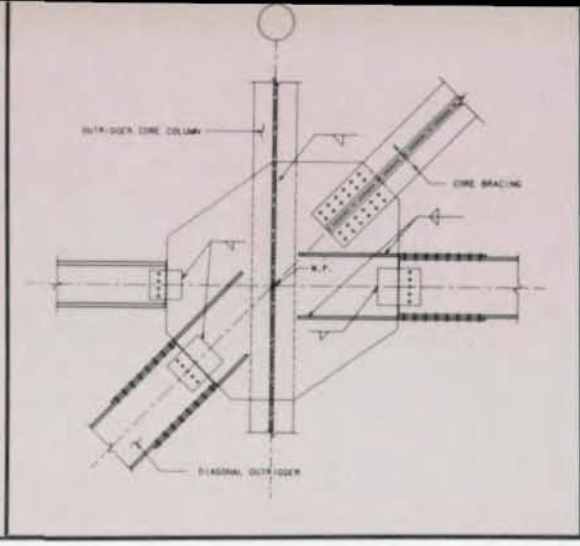


Fig. 5. Outrigger elevation *Fig. 6. Outrigger connection to exterior column* *Fig. 7. Outrigger connection to core column*

led with six four-story diagonal outriggers at each face of the core (Fig. 5) located at three points over the height of the building. The system works in a similar manner to the mast of a sailboat, with the braced core acting as the mast and the outrigger superdiagonals and verticals forming the spreader and shroud system. After various studies employing in-house optimization computer programs, three sets of eight outriggers were found to be the most efficient solution. Although simplified models showed they would be the most effective if spaced at equal intervals, optimization programs showed these outriggers could further reduce wind-induced drift without adding a single point of steel by simply modifying their spacing over the height of the building. Ultimately, the design was completed with the outside end of the superdiagonals placed at the 20th, 37th and 51st floors. The outrigger superdiagonals are connected at the exterior of the building to vertical outrigger columns as shown in Figs. 6, 7 and 8.

To reduce uplift forces on corner core columns and the outrigger columns, the design team found it desirable to concentrate most of the building's dead load on the corner core columns and the outrigger columns. This was accomplished by introducing exterior transfer trusses at the 6th, 21st and 37th floors, which span between the outrigger columns within the exterior face and thus funnel dead load into the outrigger columns to compensate for uplift due to wind pressure (Fig. 9). Uplift in the exterior outrigger columns was totally eliminated with this approach. The uplift on the corner core columns was reduced to 1,300 kips.

In developing the superdiagonal outrigger system, an intensive effort between architects, interior planners and developer was undertaken to determine that the presence of diagonal outriggers penetrating down through certain lease space at eight locations on 12 floors would not interfere with efficient layout. Interior planners made various layouts for full-floor and partial-floor tenants and concluded the inclined superdiagonal columns would not hinder the real estate leaseability.

Wind forces were generated using prevailing codes and also utilizing a force balance wind tunnel test undertaken by Cermak/Peterka of Fort Collins, Colo. It was determined that average wind pressures on the building varied between 5 psf at the bottom to 58 psf at the top. Both planar and three-dimensional static and dynamic analyses were performed for combinations of gravity and lateral loads. The period of the building was determined to be 5.5 seconds.

The lateral load resisting system was designed initially using a purely allowable stress criteria. During the optimization effort, members were increased in size, which contributed to increasing the building's lateral stiffness. Stiffness was in-

Fig. 8. Outrigger superdiagonals connected to vertical columns

Figure 9

creased and the acceptable limits of building drift (H/450) and acceleration (15 milli gs) were met. In addition, because of the vertical compatibility between the outrigger columns and core columns created by the outriggers, analyses had to be performed for gravity loads to determine the gravity load magnitude in the lateral load resisting system. This analysis was performed in steps to properly model the actual building erection and loading sequences.

Use of the optimization program trimmed an estimated 2 psf from the wind resisting system, a savings of some 15% by weight. More important were savings gained by eliminating entire components such as two interior bracing lines above the 20th floor, which greatly simplified design and construction.

Erection of the Structure Efficient

Erection of the structure was undertaken by two cranes. An FMC-1900 supported off the braced core and a FAVCO 1000 supported outside the builidng on an independent tower. This allowed the job to get done more quickly and efficiently. The larger crane, which operated from the core of the building, erected most of the structural steel. It is equipped with a hydraulic jack which jumps the mast about 50 ft at a time, whenever more height is required. A second outboard crane operating along the northeast corner of the building stocked each deck with small steel members and light materials. The erection, started in February 1986 on this 17,326-ton structure, was completed in only 11 months.

To lift the 120-ft spire to the roof of the building, the FAVCO crane is moved inboard to attain maximum capacity. The tapered cruciform spire is four separate steel sections, each 25 to 35 ft long and weighing 20 to 25 tons. The spire's steel, exposed to high winds and low temperature, was specified as fine-grained, silicon killed steel with Charpy V-Notch requirement of 25 ft-lbs. at 20° F. Before the spire can be lifted into position, hoisting equipment must be moved to the top of the structure. Current plans call for the larger core crane to lift the smaller outboard crane onto the building at the 58th floor. Once installed inboard the smaller crane will lift the larger crane off the building. The smaller crane will then complete the spire erection and will be lifted off the building by a small derrick which in turn will later be removed by an even smaller boom hoist. This procedure is scheduled for completion by February 1987.

Outrigger and Column Details

Because of the large column sizes—1,852 lbs. per linear foot (plf) for the outrigger columns, 947 plf for the diagonals and 1,852 plf for the corner columns—the team decided to use built-up wide-flange sections rather than jumbo sections with cover plates. Not only did this simplify the connections, but also it permitted partial penetration of welded splices (Fig. 10). If jumbo column sections with cover plates had been used, bolted splices of extremely large size would have been dictated. These splices would have been very costly and would also interfere with architectural finishes and elevators.

Acknowledgments

The authors would like to thank Joseph P. Denny and John J. Farrell of Rouse Associates; Robert Pearson of Morse-Diesel and Antal Partos of Site Engineer for their valuable contributions in selecting the systems on the project.

(continued)

One Liberty Place, an elegant statement about its city

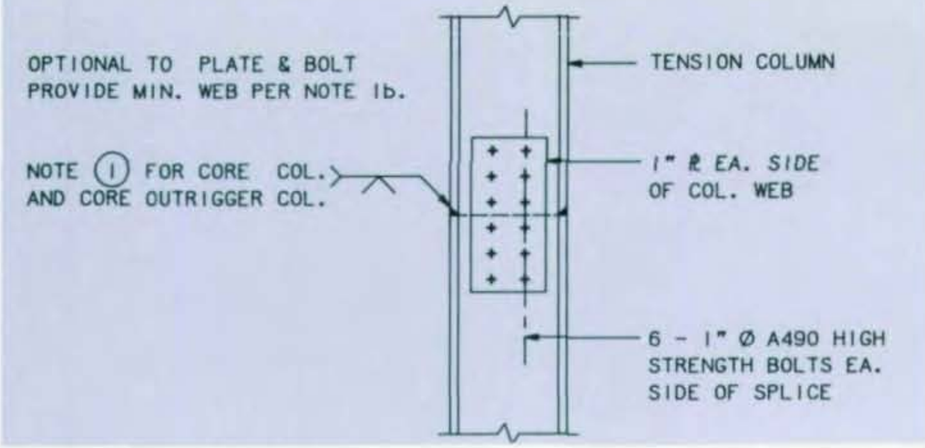


Fig. 10. Typical splice detail of tension and outrigger columns. Note—provide weld to develop greatest of following: (a) 200 kips tension minimum; (b) min. size = tf/6 + $^1/_8$"; $^1/_3$ of column capacity is tension.

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Chicago, Illinois

Structural Engineer
Lev Zetlin Associates, Inc.
New York, New York

Owner
Rouse and Associates' One Liberty Tower, and One Liberty Place, Tower Trust and Affiliate of Teachers' Insurance and Annuity Association

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General Contractor
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Steel Fabricator
Steel Structures Corp.
Bethlehem, Pennsylvania

Developer
Rouse and Associates
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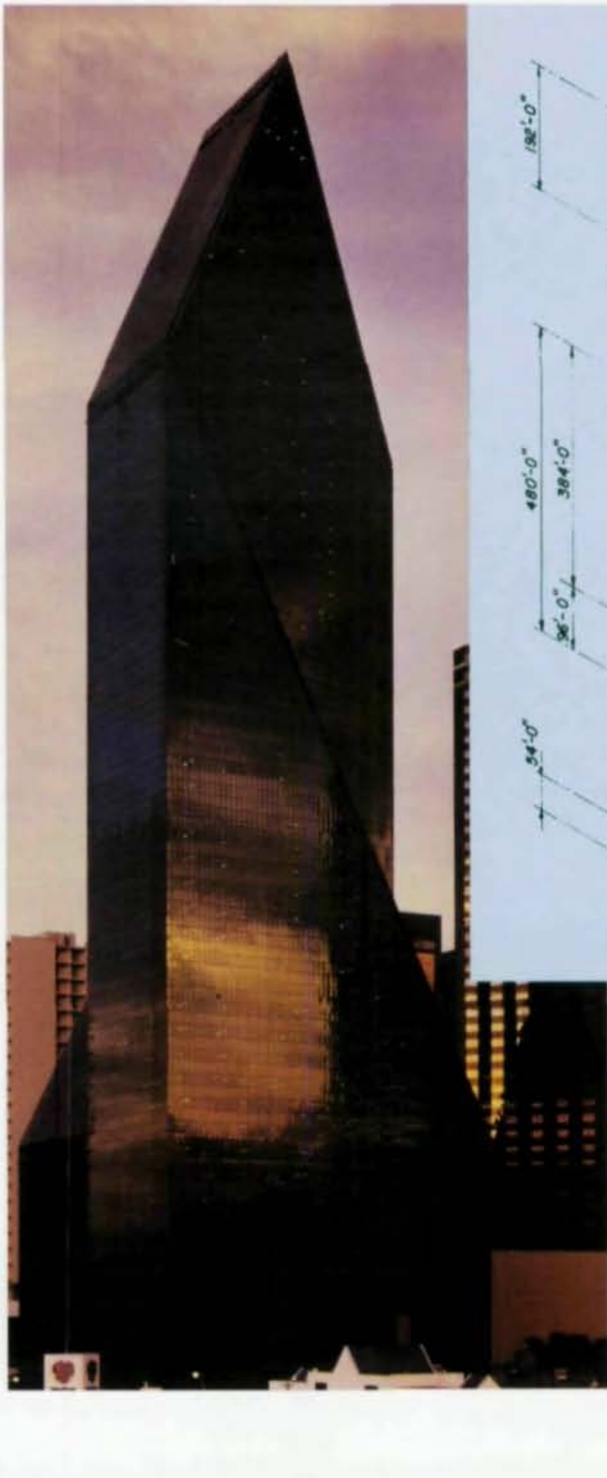
Scott Pratt is vice president of the architectural firm of Murphy/Jahn, Chicago, Illinois.

Innovative Structure Systems

ALLIED BANK TOWER

A Landmark Stands Tall—with Steel

by P. V. Banavalkar, D. Parikh and W. Ling



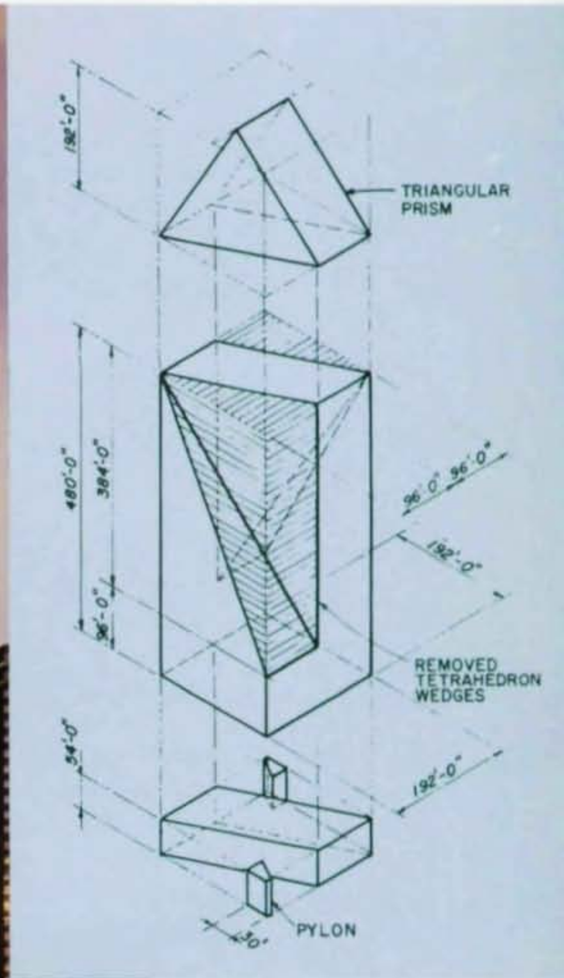


Fig. 1. Geometrical composition of building exhibits its unconventional shape

Allied Bank Tower stands 60 stories (726 ft) tall, like a giant rocket, above a plaza full of bubbling fountains and bald cypress trees in Dallas' central business district. According to critic Paul Goldberger, it is "the most compelling building in the city" and "likely to become as symbolic of Dallas as the gates of Southfork Ranch." *Engineering News Record*, in a cover story, declared, "Unconventional Shape Creates Instant Landmark."

Unconventional Building Shape

The unconventional geometrical shape of Allied Bank Tower, that makes it "compelling" or an "instant landmark," is no doubt indescribable in simple terms. The geometrical composition of the complex shape, however, demonstrated in Fig. 1, can be described thus: First, a rectangular block 192 ft square in plan and two and a half times in height (2.5 × 192 = 480 ft) is carved out of two 384-ft (2 × 192) high tetrahedron shaped wedges from two opposite sides above 96 ft (½ × 192) height. The removal of such wedges gradually changes the square plan at the bottom to a rhombus-shaped plan (96 ft × 192 ft) at the top. The rhombus-shaped plan is then capped by a 16-story (192-ft) high skewed triangular prism pointing upward. The combined 672-ft high block is pedestalled off a 4-story (54-ft) high rhomboidal block that matches the rhombus shape at the top. The overhanging corners at the base are supported on 30-ft wide pylons. The resultant unconventional shape (Fig. 2) provides large, 4-story high triangular open entry spaces to the rhombus-shaped building lobby, vertical and sloping exterior facade and a distinctive top.

Figure 2

EVOLUTION OF STRUCTURAL SYSTEM

Factors Affecting Selection

The choice of the most economical structural system was not only dictated by the geometrical form of the building but also by the need to contend with other architectural constraints.

The column-free, right-angled triangular areas on either side of the basic rhombus-like floor plan had to be supported only at their corners. Forty stories of office space in these triangular areas demanded 156 ft and 95 ft of free spans interspaced by 30-ft wide corner pylons. Above the 13th floor, one side each of these areas start sloping in opposite direction with 2 to 1 vertical to horizontal slope. It was therefore apparent at the outset that this building, when subjected to lateral wind loads, will not only demonstrate the unsymmetrical bending mode but also will be prone to twisting. The column-free, long-span space at the base, the desired torsional stiffness against the lateral loads and the architectural profile of the building itself were decisive in determining the structural system.

Various Options Considered

Because of reduction in the floor area and the changing core plans with height, coupled with possible discontinuity at the base due to three basement levels, an efficient core bracing, either in the form of braced frames or concrete shear walls, was not suited to the efficient lease space requirements.

A perimeter modified steel tubular structure with closely spaced columns was also considered. However, the economics and aesthetics of such a system with the transfer girders at the base either in the form of 4-story high trusses or vertical Vierendeel girders made the solution unattainable. Needless to say, the profile of the structure with transfers at the base rendered all concrete or a perimeter composite concrete structure totally impractical.

Major Features of Final Choice

Finally, an innovative economical structural system evolved. A perimeter "trussed frame" capable of performing dual function of free span at the base with a desired lateral load resistance provided the optimum solution. This system gave 40-story deep "mega trusses" on two sides (Fig. 3) to economically bridge 156-ft spans on two opposite sides and 8-story deep trusses to bridge 96-ft spans on the other two opposite sides.

But the architectural constraints needed further innovations to the system. Trussed exterior frames were to be set back 3 ft from the skin of the building to clear the curtain wall. And, in order to be in harmony with the overall governing geometry of the building, the truss diagonals had to be

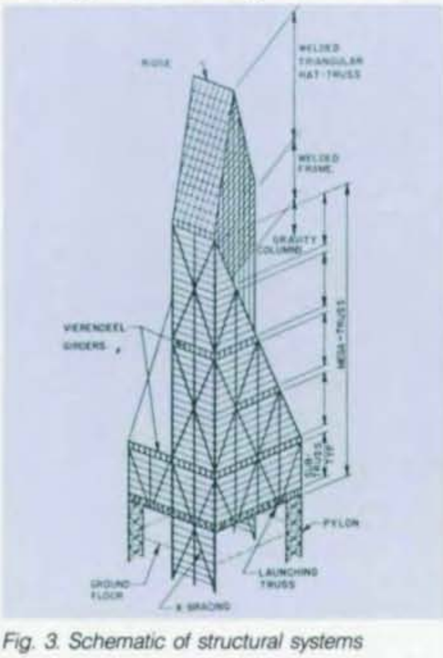


Fig. 3. Schematic of structural systems

sloped at the same slope as the ridge of the tower—namely 2 vertical to 1 horizontal. With the restriction of the slope and 12-ft floor height dictated by required lease space, the layout of the trussed diagonals resulted in the intersection with the columns at mid-height of a floor. As opposed to the conventional truss action, it was not possible to transfer the unbalanced horizontal components of axial forces in the diagonals directly to horizontal floor members. Therefore, a full story-deep Vierendeel girder (Fig. 3) designed to resist the moments caused by unbalanced horizontal components of the diagonals was introduced at every eighth floor. The mega truss then got subdivided into 7½ stories high "sub-trusses" between Vierendeel girders.

Also, to fulfill the function of spanning large distances for gravity loads, trussed exterior frames were designed to provide lateral wind resistance also. However, these frames, located only in the vertical sides of the building, were not forming into a closed exterior tube. Only a pair of trussed frames, angle-shaped in plan in two diagonally opposite building corners, were available to resist wind loads from ground to 45th level. One leg of each such angle-shaped frame reduced gradually above 13th level, due to the building slope, and vanished completely at the 45th level, leaving only a pair of parallel trussed frames on two opposite sides. Such parallel frames near the 45th level could resist wind from one direction only, and if not properly interconnected the pair of angles below would twist and warp. To satisfy this dual function of providing wind resistance in both directions and minimize warping of angle frames, the moment frames were introduced all around the triangular prism above the 45th level, named "welded triangular hat-truss." The transition zone between the triangular hat-truss and the trussed frame was provided by extending moment resisting welded frame nine floors below to the 36th floor. The hat-truss successfully interconnected the angle shaped frames and prevented them from warping. In addition, it allowed simpler framing in upper floors which kept reducing to almost nothing at the 59th level.

Response to Construction Needs

Forty-story deep mega trusses or 7½-story deep sub-trusses were able to span large distances in completed form, but the question of erection still remained. To start out the construction above open spaces without 4-story high temporary shoring, the floor directly above the open spaces (i.e. 5th floor) was converted into a mechanical floor during planning stages and full story-deep trusses (named "launching trusses" in Fig. 3) were provided all around the perimeter of this floor (Fig. 4). These launching trusses were designed to support the erection loads of eight floors above; so when the Virendeel girders at the 12th floor were erected, the 8-story deep sub-trusses were formed. The sub-trusses along with the launching truss would then carry erection loads of the next eight floors above until the second set of Virendeel girders were completed and the process of building up mega truss in stages continued. The structural analysis, explained later, thus had to take into account the staged construction and sequential loading effect.

Other Structural Elements

Within the fifth floor, story-deep transfer trusses were also provided, spanning be-

Figure 4

tween interior columns along rhombus and exterior launching trusses, to support columns within 40-story high triangular areas directly above open entry spaces. The column spacing in these areas was reduced to about 24 ft, permitting simple floor framing.

Below the 5th level, the 30-ft wide pylons on two opposite corners were the only elements available to transfer entire north-south wind shear from above. Heavy built-up columns and W14 X-braces were required to transfer this shear. The gravity load from 156-ft span mega truss was sufficient to balance the uplift caused by the overturning moment due to wind loads. Practically no net uplift was found at foundation level.

STRUCTURAL ANALYSIS

Computer Model

A three-dimensional computer model of the building was analyzed in McAuto STRUDL program. It required 2,038 joints, 3,724 members and 250 finite elements. In addition to all the exterior members shown in Fig. 3, in-plane truss members were included at the 45th level, where sloping rafters of hat-truss intersect Virendeel girders. Finite elements represented the in-plane diaphragms at this level and at all floors with Virendeel girders (i.e. at 12, 13, 20, 21, 29, 36, 37, 44 and 45) as well.

Wind Loads

It was apparent at the outset that the main structural frame design for lateral wind loads based upon the prevailing building codes needed to be further substantiated by wind tunnel studies. The initial sizing of the structural members resisting lateral loads was done by using wind loads from ANSI Building Code with Exposure B.

The dynamic properties, such as two lateral sway and torsional mode shapes, along with three fundamental periods about the orthogonal axes, were used in a force balance model tested at the University of Western Ontario. The lateral shears, overturning moments and torque determined from the force balance model were used to optimize the structure. Apart from assuring the strength and stability of the building, efforts were made to increase the effective mass of the building by adjusting the vibrational mode shapes, so as to improve the acceleration at the top occupied floor. The final building analysis for wind was done for both ANSI Building Code with Exposure B and loads determined by force balance model.

Gravity Loads

As explained earlier, the structure had to

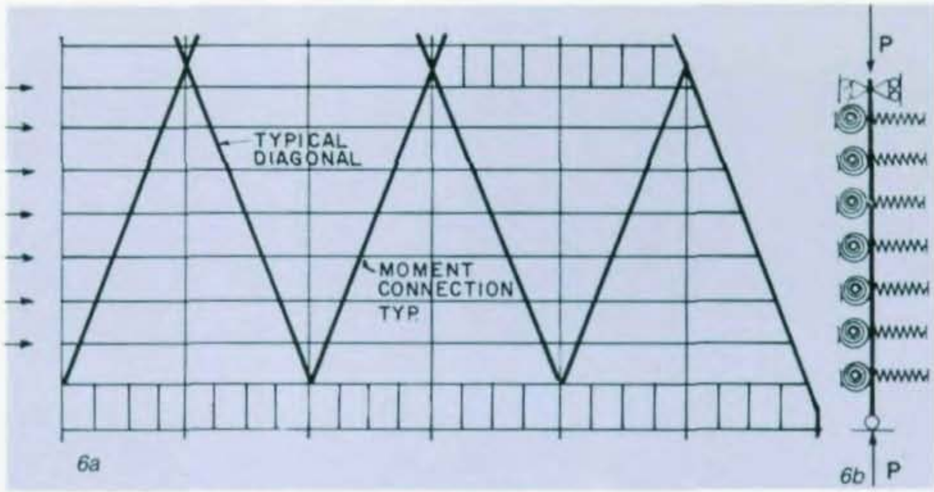


Fig. 6a-subtruss; 6b-model for diagonal buckling analysis

be analyzed for sequential loading to correctly represent the staged construction of mega trusses. Gravity loads were therefore broken down into various components, viz. weight of steel and metal deck, concrete floor slab, curtain wall, partitions, ceiling and mechanical and live load. Sequential load analysis for gravity loads was then performed for 13-story, 21-story, 29-story, 37-story, 45-story and 60-story high models with the help of INACTIVE commands. Appropriate amount of loads (steel structure, concrete floor slab, etc.) based on anticipated construction sequence were applied to each model.

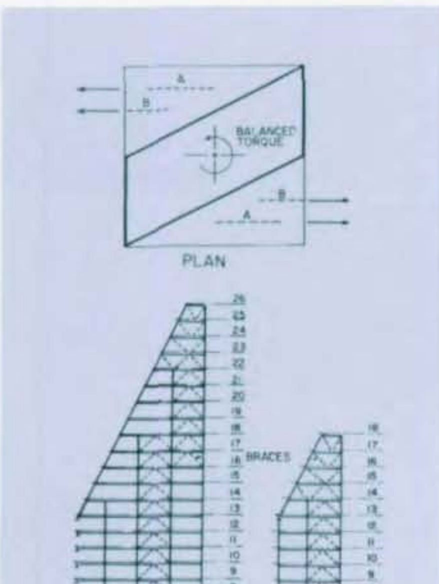


Fig. 5. Temporary construction braces

Loading Combinations

To determine the worst design condition for each member of the structure, 35 individual loading conditions and 76 loading combinations were required. Each member was then checked for strength adequacy by using STRUDL code check.

SALIENT DESIGN FEATURES AND DISCUSSION

Shape, Construction and Design

The impact of shape on design for ease of construction manifested itself in three areas; the launching truss, temporary vertical bracing and in-plane bracings above the 45th floor.

During the erection of the structure, the compression chord of the launching truss required positive bracing. Two options were considered. An in-plane truss around the perimeter with required strength and stiffness was designed to offer the bracing. The other option was to pour concrete slab at the 6th level, tying the adjoining sides for in-plane stiffness. After the cost evaluation, the contractor opted to pour the concrete slab.

Unlike regular symmetrical buildings, the structure showed a tendency to twist about the vertical axis under gravity loading due to the presence of sloping rafters on two opposite sides. The resistance to this twisting was provided by the sub-trusses in the completed form. However, during construction, two temporary rows of vertical bracing were provided normal to sloping sides to prevent the twisting (Fig. 5).

The sloping top of the structure starts at the 45th level. Above this level, in-plane floor trusses between two opposite ends of welded hat-trusses were designed to restrain the structural frame from unwarranted sway. The noted special behavior of the structure during construction was anticipated and catered for during the design phase of the building. The satisfactory performance of the building during construction further substantiated the predicted behavior.

Function of Sub-trusses

A sub-truss element between the intersection of diagonals consists of moment resisting welded frame, which provided three major functions (Fig. 6a).

1. It acted as a frame between the diagonal intersection to transmit wind loads applied at each level to the panel points of mega truss.
2. Through the interaction with the floor diaphragm, the welded frame in the sub-trusses provided lateral restraint in the final form to the tendency of structure to twist above the 13th floor (see section on "Shape, Construction, and Design").
3. The entire sub-truss moment-resisting frame provided a bracing sandwich to prevent the buckling of the compression diagonals of the mega trusses. A buckling analysis of the diagonal braced by the shear resistance of the frame element was performed (Fig. 6b). Details of this analysis are beyond the scope of this article.

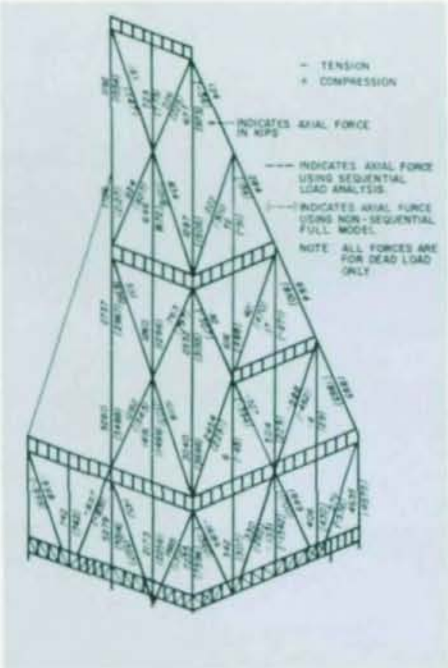


Fig. 7. Results of sequential analysis for sustained dead load

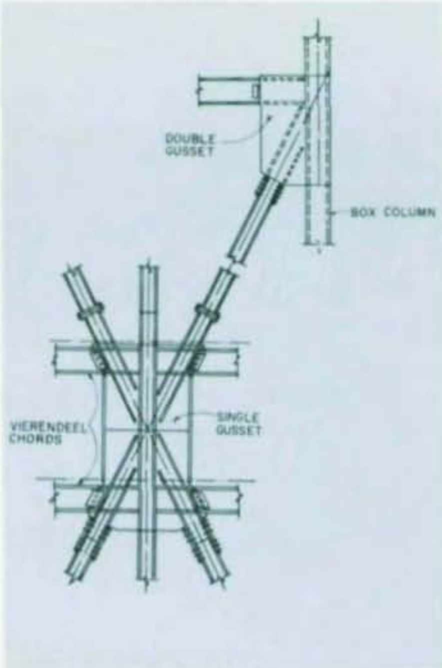


Fig. 8. Connection details

Sequential Load Analysis

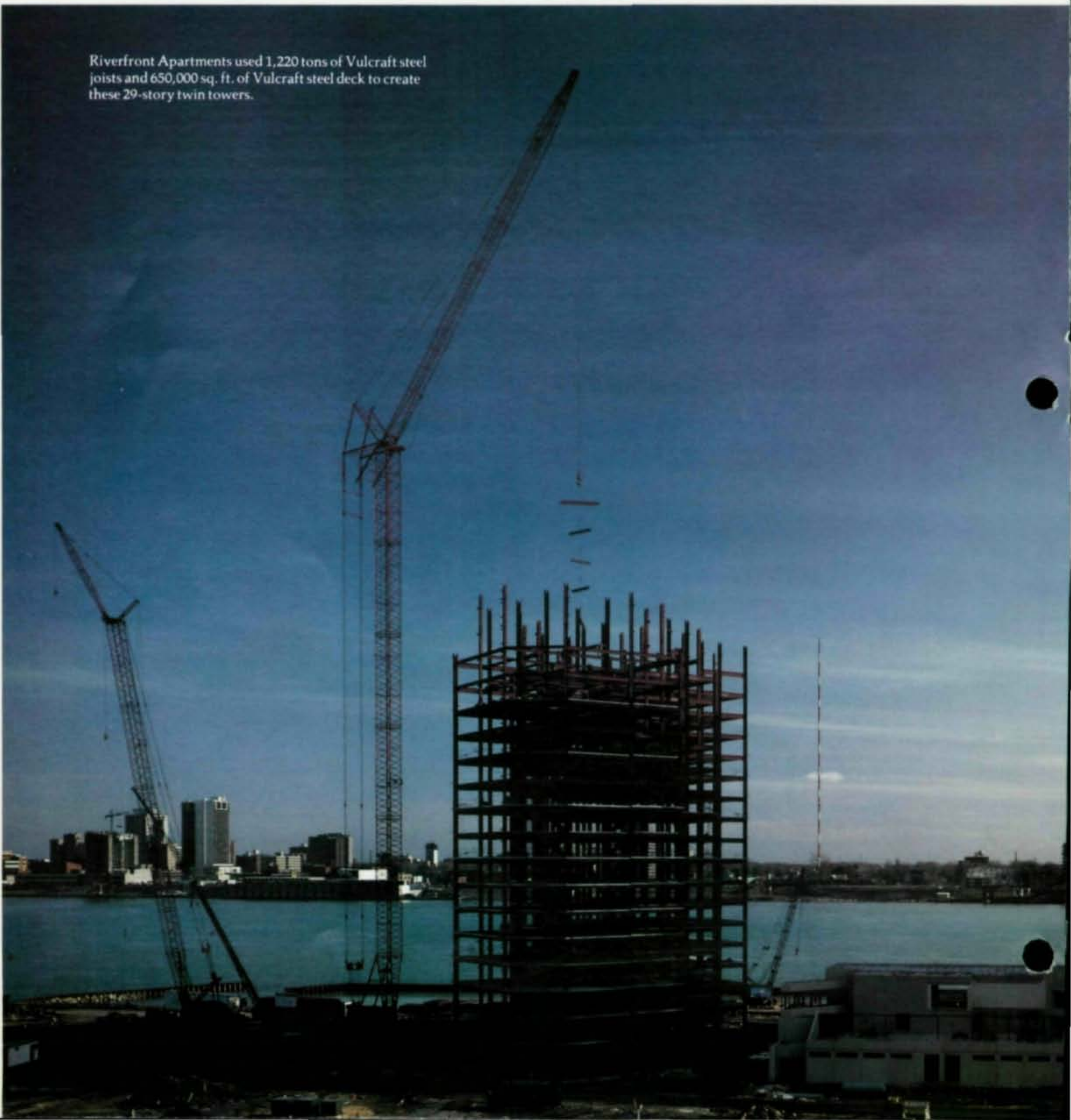
The sequential load analysis was very essential for this structure. The principal truss members carrying mainly axial forces and having insignificant capacity of redistribution of loads required accurate prediction of design connection forces. It should be evident that the forces in the launching truss, found from proper sequential load application, will be higher than those determined from a full-height model. The same conclusion is true for lower level diagonals as well. However, the reverse is true for upper level diagonals (Fig. 7). The error without sequential load analysis could be as much as 20%. The load redistribution capacity of triangular hat truss would have been overestimated without sequential load analysis.

The prediction of in-plane vertical deflection of the mega truss is very crucial from the standpoint of performance of the curtain wall, as the unwarranted cambering of the mega truss would be detrimental. The satisfactory installation of the curtain wall further signifies the value of the sequential load analysis of the building. The sequential load analysis also helped in the proper axial shortening adjustment between interior and exterior columns.

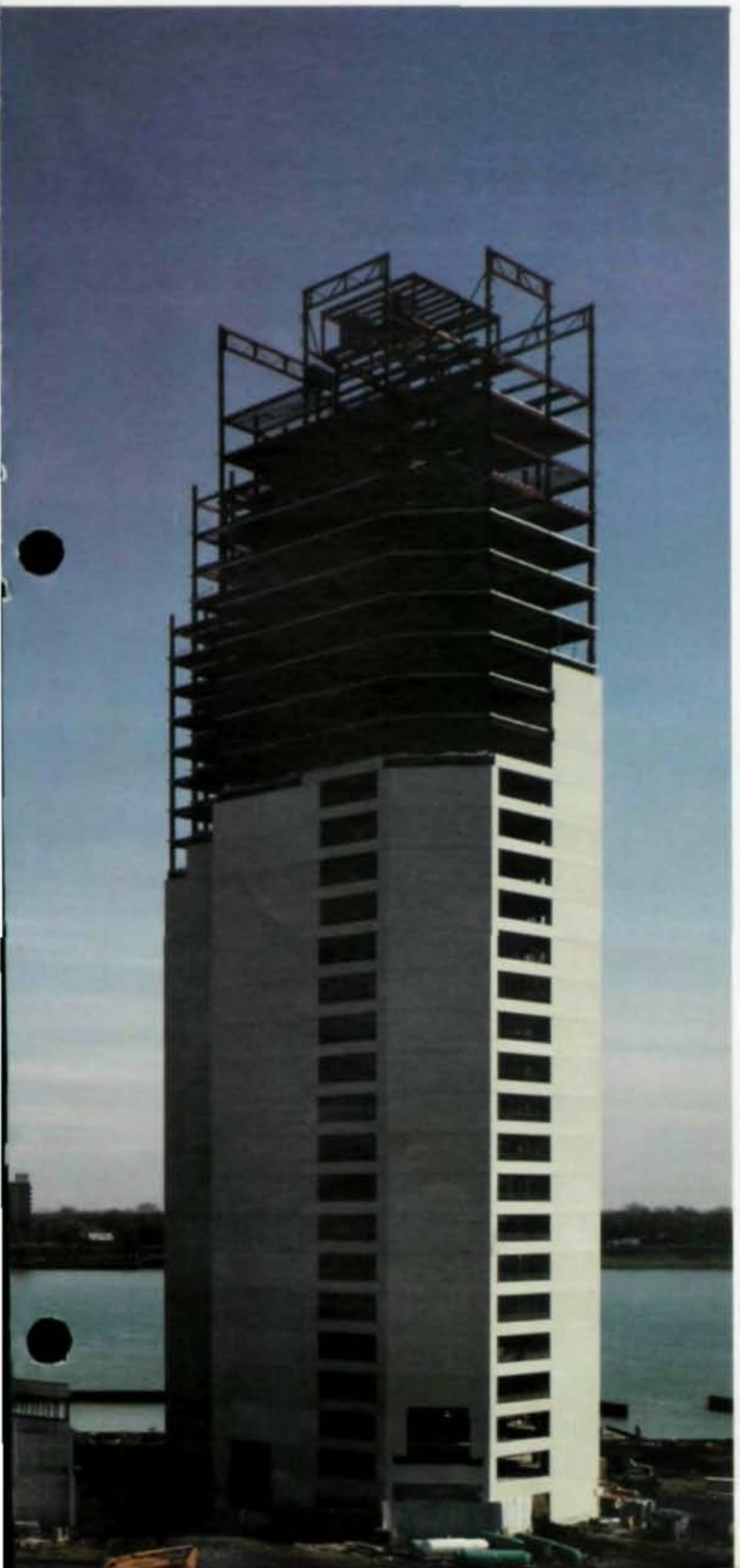
Connection and Fabrication

Both A36 and 50-ksi material were used for the project. All diagonals in the mega truss were W14 rolled shaped. The maximum weight of the trapezoidal column at the corner of 156-ft span using 8-in. thick

IN DETROIT, VULCRAFT
IN THE SKYLINE AND



Riverfront Apartments used 1,220 tons of Vulcraft steel joists and 650,000 sq. ft. of Vulcraft steel deck to create these 29-story twin towers.



plates was 2,450 lbs./ft. Two-sided gusset plate details were used at the intersection of diagonals with the box columns, whereas single-gusset plate with stiffeners matching wide-flange shape diagonals and columns were provided at the intersection of diagonals (Figs. 8 & 9).

The connections at the intersection of diagonals were analyzed by finite element method, with the octahedral stresses remaining below the allowable stress level. Connections were designed both for strength and stiffness.

Discussion of Wind Loads and Stiffness

Considering only unidirectional wind, the wind loads employing dynamic properties of the building were found to be slightly higher than those given by the ANSI Building Code. However, the combined re-

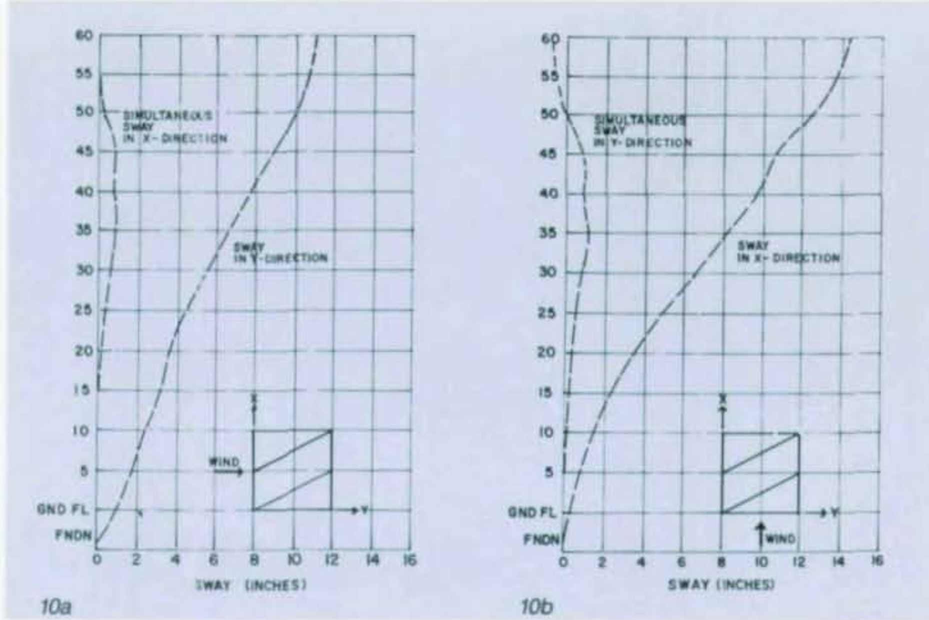


Fig. 10a and 10b Wind sway

Fig. 9. Connections

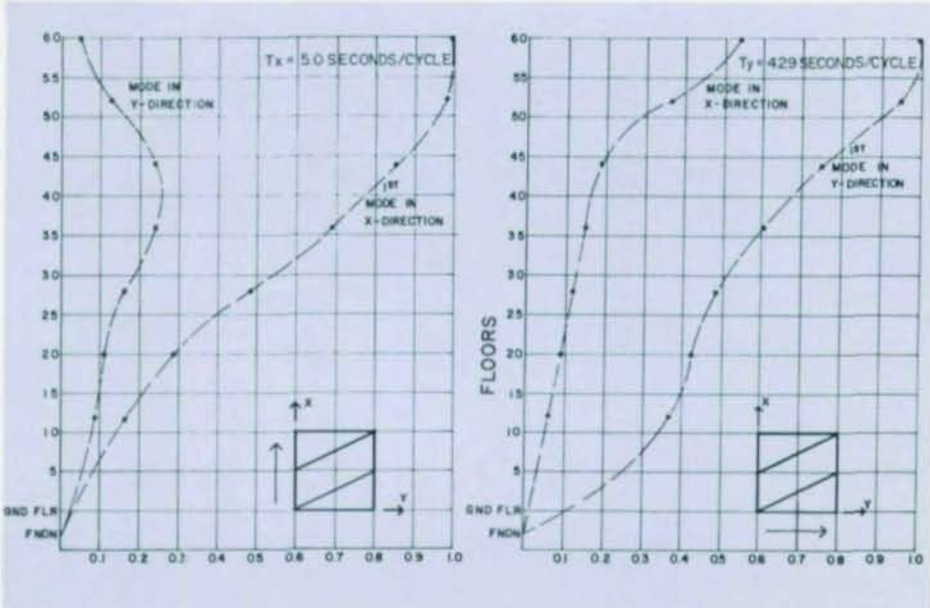


Fig. 11a. 1st mode shape, x-direction

Fig. 11b. 1st mode shape, y-direction

sponse to the orthogonal axes of the building coupled with the torsional loads controlled the design of members. Insignificant twisting motion with ± 1 in. displacement normal to the direction of load at 45th floor substantiates the effectiveness of hat-truss in increasing the torsional rigidity of the structural frame. The lateral sway of the building for the ANSI Building Code wind load is shown in Figs. 10a and 10b.

Apart from the structural wind loads, the occupant's comfort at the top floor of the building was of major concern. Also, because of the orthogonally coupled vibrational mode shapes of the building (Figs. 11a & 11b), it was felt the force balance study results have to be substantiated by dynamically precise aeroelastic model. A six-lumped mass model with appropriate dynamic properties was therefore tested in the wind tunnel. Because of the large shear distortion of 30-ft wide pylon at the base and diminishing slope at the top due to large stiffness of triangular welded hat-truss, the mode shape in Y direction was found to be beneficial in increasing effective resisting mass of the building against dynamic wind loads. For 1% structural damping, a conservative assumption for this structure, the peak acceleration at 40-ft radius at the 58th floor (the last occupied floor) was found to be in the lower 20's for once in 10-year wind occurrence.

Miscellaneous

The floor construction consists of composite 50-ksi strength steel beams supporting 5½-in. thick normal weight concrete slab inclusive of 3-in. deep metal deck. In lieu of welded wire fabric, mesh fiber reinforced concrete was used to speed up floor placement. Additional reinforcement was used at Virendeel floors along with floors 13 and 45, where the structural profile changed the shape of the exterior facade.

The plaza with bubbling fountains was supported off a composite metal deck and concrete slab on steel beams.

With three basements below plaza level, 50-ton capacity rock was available to support the tower columns just below the third basement level. The maximum design column load was in the range of 14,000 kips. Shallow drilled piers bearing on the rock were used. Even for the most severe direction of wind there was practically insignificant uplift at the base of the columns due to the design dead load transferred from large spans.

The innovative structural system, located primarily on the exterior faces of the building, was found to be very economical for this height and shape of the building. With only 22 psf steel weight, it satisfied the owners' need for an economical structure—and still fulfilled the asthetic and functional requirements remarkably well. A conventionally designed perimeter tubular structure with equal stiffness and serviceability performance would have required 6 psf of additional structural steel. The present structural system saved nearly 3,500 tons of steel over a conventional design. The interior of the building is free of any major structural elements and gave the architects freedom in laying out varying core plans for floor areas. A floor-to-floor height of 12 ft only (as against 13 ft generally used for this type of office building) was thus made possible. Architects I. M. Pei & Partners also studied the presence of diagonals near the building exterior, along with the alternate tubular scheme of closely spaced columns, and found the trussed scheme presented the least obstruction to tenants' exterior view. This can be appreciated by the fact that only one diagonal was present between columns spaced at 48 ft, resulting in an average of 24 ft of column spacing compared to 10 ft to 15 ft spacing generally found in a tubular scheme.

The successful completion of the project in record time speaks for the coordinated efforts of the owner, contractor and the design team members. □

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HCB Contractors
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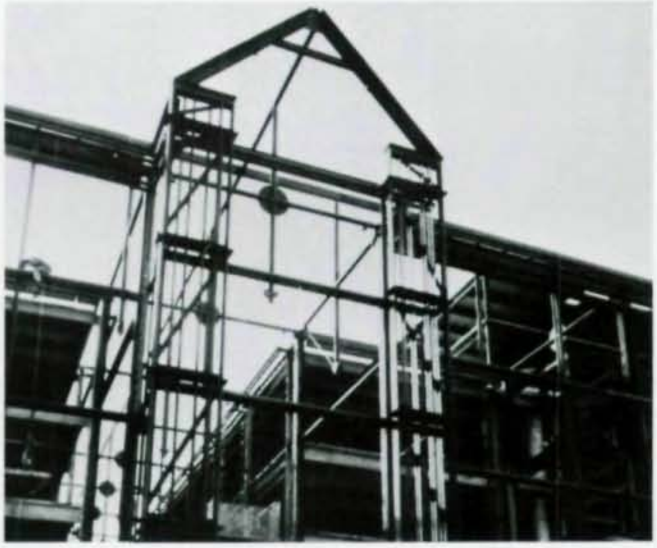
Fast-track Design

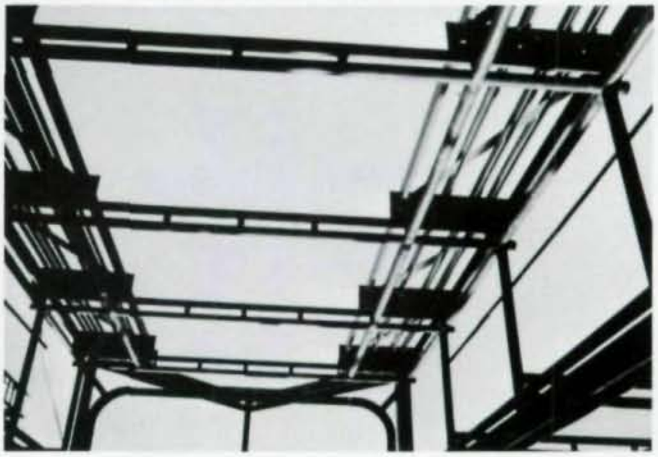
GALLERIA OFFICENTRE
A Study in Gothic—and Steel

by Ken Neumann and Robert Cooper

The tremendous success of the original Galleria Officentre in Southfield, Mich. led to expansion plans that will nearly quadruple its size. The opening phase, constructed in 1981, was 250,000 sq. ft of four-story office space. Before this space could be completed, another 250,000 sq. ft was under construction. Now, construction is underway for another 350,000 sq. ft, part of an expansion plan that will total 1,350,000 sq. ft. Steel framing has contributed to the success of each phase.

The architectural challenge of the original 4-story project was to create a modern office building on a long, narrow site bounded on one side by a major highway and on the other side by a planned 50-acre residential complex. The solution was to use a multi-faceted reflective glass curtainwall on the expressway side and a brick facade on the residential side, where brick was planned as the key element. With the demise of the residential plans, the developer of the Galleria was able to purchase the site and convert it to a planned office complex expansion. A new architectural challenge arose—how to in-





Handsome, Gothic-styled "spline" (above) connects new and existing buildings. Photos (r.) show spline's steel-framed construction.

tegrate the brick facade into the expansion area.

The result is a complex using a combination of the two materials, glass and brick, that provide the uniformity and variety of a village where each structure has a distinct look and yet has a common fabric.

Each building features double-glazed, silver reflective glass accented with areas of copper metallic reflective glass that yields a reddish-pink hue. Clear glass is used in the atrium areas. A narrow band of dark blue glazed brick creates an accent, one not noticeable from a distance but used for visual interest up close. The brick is medium to dark brown.

Steel Framing Throughout

Steel framing has been used throughout the project because of its speed of construction and economy as opposed to other framing materials. It is difficult to achieve the attractive open and long floor spans with any other material. Steel framing was best suited to handle the numerous corners throughout the project, used to provide premium corner offices, and the complex variety of curtainwall attachments.

The original four-story Galleria employed a typical 24-ft wide by 32-ft deep bay, three bays from front to rear of the building. In the direction parallel to the expressway, the building was 28 bays long, divided into three offset sections. The addition was attached to the original building by a high, open atrium of glass that features open walkways covered with polished stainless steel strips. The addition repeated the same offset bays and deep spaces.

The current expansion of the Officentre is detached from the original building, but tied together with a tree-lined walkway that features arches of 8-in. dia. pipe every 20 ft. These 8-ft high, 20-ft wide rectilinear arches provide not only a visual link between the buildings, but also frame the view of either end for the pedestrian.

The new structure, with 350,000 sq. ft on five levels, features a full-height atrium through the middle and across the north end. The central atrium aligns with the arched walkway to the original complex, and lends the appearance to some of a gothic cathedral. High, long and narrow, the atrium features light roof framing of pipes, rods and tubes. The atrium floor uses brick pavers accented with sections of polished granite and marble in three colors. Brick planters and trees are spaced throughout.

The east section of the building reflects the glass curtainwall of the existing building, along with numerous facets. The west section of the building uses the brick curtainwall of the rear of the existing building. This integrates the new structure with the old and sets the architectural theme for the entire complex.

Typical bays are 20-ft wide and 36-ft deep, and each side of the building is three bays deep. Beams spaced at 10-ft centers are supported by columns and 20-ft beams serving as girders. Framing in this direction minimized deflections and concrete slab cracking in the negative moment regions. Floor-to-floor height is 12 ft-8 in., with ductwork running below the beam framing. The floor slab is 18-ga., 2-in. composite metal deck used in the three-span condition, supporting a slab of 4-in. normal weight concrete. Beams and columns received spray-on fire protection.

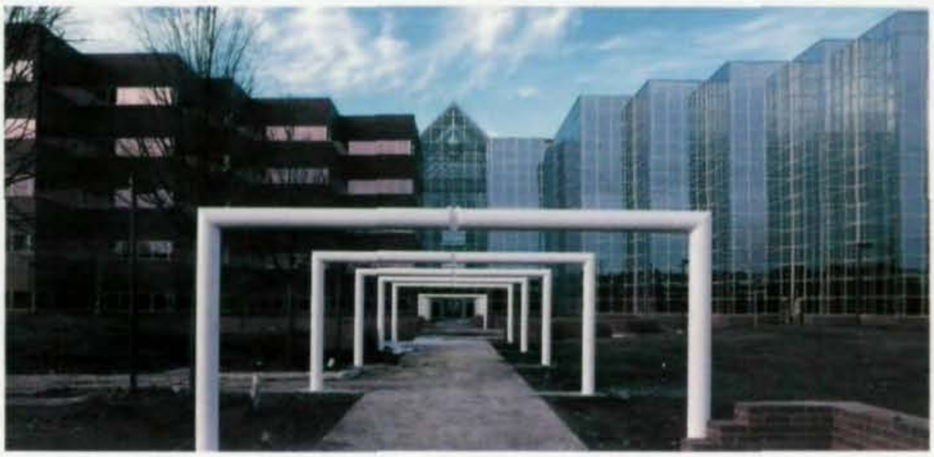
Lateral loads are taken by a series of narrow vertical trusses placed at stair towers and elevator cores in the E-W direction and moment frame in the N-S direction. The systems did not interrupt any of the floor spaces or exterior views. Angle bracing was adequate because of the limited building height.

The Atrium—Gothic and Steel

The atrium roof, part glass and part standing seam metal panels, is supported by a series of pipe and rod cross-frames on a tubular ridge beam and mullion system. The pipe frames, spaced 20-ft o.c., span 20 ft across the atrium. A horizontal and vertical pipe 8 in. in dia. forms a cross, and a 2-ft radius of plate helps form the intersection of the cross. Rods 1.5 in. in dia. run from the ends of the horizontal pipe at the supports to the bottom of the vertical pipe, forming a counter-statement to the roof itself. The stiffness of the vertical pipe provides stability to the rod/pipe connection point, which is not laterally braced. A rectangular tube forms the ridge beam, which in turn supports the glass mullions and roof panel framing. Pipe and rod members were over-sized for visual effect.

Because of shipping size limitations, the steel fabricator shop-assembled the horizontal pipe with the lower pipe vertical, and used mechanical tubing of the proper outside diameter to make a sleeve fitting inside the 8-in. pipe for the upper vertical pipe section. A similar sleeve detail was used for the end support conditions at

Above, existing building from new parking lot.
Buildings complement each other in both form and function.



each end of the pipe. Rods were attached using a threaded rod clevis detail.

Another difficult framing area was the support of the end curtainwall at the north atrium. This atrium will serve as a joining part for a future building still in planning. Therefore, it was necessary to provide independent support of the five stories of glass. This was done with a Vierendeel truss system made of pairs of 6-in. dia. pipe. These pipe trusses, with the chords spaced 12 ft-8 in. apart and connected at 4-ft centers using 4-in. dia. pipes, span 36 ft column to column. A vertical grid of similar pipe trusses is spaced 12 ft o.c. The Vierendeel resists not only the lateral wind load applied through the 4-ft square grid of glass mullions, but also the outermost pipe picks up the vertical loading from the glass curtainwall itself, inducing torsion into the Vierendeel. The columns are unsupported laterally for the full five-story height in the direction parallel to the curtainwall, and are braced using struts tying back to the main structure at the third and fifth floor levels. Because the pipe trusses support only the curtainwall, no fire-protection was required. The columns and struts are fireproofed to the required height, and are concealed with a white pre-fabricated Fiberglass cover. The pipe trusses are painted a matching white.

On the north wall of the north-south central atrium, a precast concrete arch is supported 60 ft in the air by the steel framing above. The arch provides a high focal point to the atrium, and also adds to its gothic appearance. Trolley beams were also placed inside at the atrium near the intersection of roof and wall to support a travelling window-washing unit. These beams were curved at all atrium corners to allow for uninterrupted movement of the unit.

For the balance of the building, the glass curtainwall was attached to the steel framework using conventional means. The brick facade was supported by a horizontal angle attached to 4-in. structural channels at 2 ft o.c., which in turn were attached to the floor beams and a horizontal channel sub-girt system suspended using sag rods from the beams above. Lateral brick wall loads were taken by these channels, which were connected to exterior columns on 20-ft centers, and the floor diaphragm. The close column spacing in this direction facilitates the use of this method, which eliminates the torsion on the exterior floor beam from the brick support angles, and also minimizes the effect of floor beam deflections on the movement of the facade.

In the north atrium, open elevator lobbies at each floor level are faced with a serpentine brick wall spanning some 30 ft. A straight beam was used to span the dis-

Stargazer Symbol for Park

"Stargazer," a 30-ton steel creation by Michael Hall, is believed to be the largest outdoor sculpture in Michigan. About 30 by 40 ft in area and 45 ft high, it deals with a series of ideas which evolved from Hall's work over the last 20 years. Hall, sculpture department head of Cranbrook Academy of Art, sees the work relating to its site and to the state in general.

Tim Hill, owner of Hill Gallery, who arranged placement of Stargazer, suggests the sculptor sees the structure of wide-flange beams as similar to building supports or shipbuilding frames which hold up flat, triangular plates representing a building's skin or a ship's hull.

The plates, arranged in patterns starting in pure mathematical order, evolve into random relationships. The change in pattern represents both order and abstract thinking. The gaps in the steel plate screen suggest patterns in the sky, trade routes or quilt patterns.

Sited near a main walkway, Stargazer, with its grey-blue color suggestive of a Michigan sky, is positioned to the sun's movement. It has become a symbol of Galleria Officentre.

tance, with the brick supported by a plate burned to the proper curvature. This plate tied back to the beam using two angle struts and a vertical 4-in. structural channel, 2 ft o.c.

Connected to the north atrium is a single-level area which serves as a service commons for both commercial and food service functions. This area is also steel-framed with beams and a 1.5-in. steel roof deck. All steel framing and the decking received spray-on fireproofing in this area, eliminating the need for a fire-rated ceiling. Because of the lightweight steel framing and a good bearing strata, it was possible to use simple spread footings to support the entire structure.

Project Schedule

The project was designed and constructed on a fast-track basis. Design began in early 1985, with the steel contract let in March. Steel erection began with the completion of the foundations in early August. One quarter of one side of the building was erected, then a second raising crew was added to work simultaneously on the other side. Two thousand tons of structural steel were erected, substantially complete by the end of October. Infilling of the atrium framing followed and the building opened to its first tenants in November 1986.

Next in the project will be a twin five-story building, constructed opposite hand and kitty-cornered to this building. They will be connected by a central steel-framed rotunda at the west end of the north atrium. This rotunda will serve as a focal point for all three future buildings and the existing building, and as the center for all commons service facilities. Future plans for the project call for two towers eight to 12 stories high, connecting to the open ends of the atria in the lower five-story buildings. The facades of these buildings will also feature the pattern of alternating brick and glass.

As now planned, future buildings will be framed in steel. The economy and open spaces of steel have helped make 850,000 sq. ft of space successful, and the same should hold true for the remaining 1,000,000 sq. ft of the project. □

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Robert Cooper is president of the structural engineering firm of McClurg Associates, Pontiac, Michigan.

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ADVANCES IN BRIDGE DESIGN AND CONSTRUCTION

by Clellon Loveall

During the 50s and 60s structural steel enjoyed a very favorable position in the bridge market. The per-ton price of steel was low. Steel beams were able to compete with concrete, both prestressed and cast in place, in the short- to medium-span range. And for spans over 100 ft, steel knew no competitor in most states.

With the interstate program fostering an expanding bridge market, things looked rosy for the steel industry. But conditions were not destined to remain the same. Just as in all fields in a free economy, if the potential for profit is available, manufacturers will compete for it. In many states, the precast, prestressed concrete industry grew to compete both for short- to medium-span bridges as well as for buildings. Reinforced concrete had been an established building product for many years, but it had limitations on span length. And its need for false work limited its use over traffic and streams. By the mid 70s, escalating labor costs and aging production facilities had combined to raise the cost of steel to a point where it had lost virtually all the short- to medium-span range market to concrete. This, coupled with problems cropping up with fatigue of cover-plated, rolled beams and some details on welded girders, served to depress the short-span market for steel girders.

For a long time, steel had a virtual lock on spans over 100 ft. But this market also came under attack. After World War II, steel was very scarce in Europe. Since almost all major bridges were destroyed or damaged, there was a need for rapid and economical long-span bridge construction. To meet this need, the prestressed concrete segmental bridge was developed. Since this type of construction demonstrated considerable success in Europe, it was natural it would come to America. With some very aggressive marketing on the part of certain consulting engineers and a requirement by the FHWA that alternate designs be considered for all major bridges, prestressed concrete segmental bridges grabbed a significant share of the long-span market also.

This is not to say the steel industry was stagnant. Constant research by the industry, the states and the federal government sought ways to improve the steel bridge.

Load Factor Design

Prior to 1971, steel bridges had been designed primarily by the working stress method. The method was easy to use and served the industry well. In the 1971 interim specifications of the *AASHTO Standard Specifications for Highway Bridges*, an alternate method for the design of simple and continuous beam and girder steel bridges of moderate length was introduced. This, called the Load Factor Design Method, made use of two techniques to improve the economy of steel bridges. Plastic stress distribution could be employed when calculating the strength of a compact section and the live load safety factor was more of a constant for all span lengths. This method, while accepted by a large majority of the states by ballot, was slow to catch on, but it has been gradually gaining more widespread use. At the present time, two thirds of the states design some or all of their steel bridges by Load Factor Design. In Tennessee, five major river crossings have been designed and let to contract with alternate designs since 1981 with steel welded plate girders competing against prestressed segmental concrete. In all five cases, the contractor selected the steel alternate as his low bid. All five bridges were designed by the Load Factor Method, which contributed greatly to their competitiveness.

Autostress Design[1]

In addition to the two previously mentioned items, the AASHTO load factor design method also permits moment redistribution in compact sections by allowing the negative moments over supports determined by elastic analysis to be reduced by a maximum of 10%, if accompanied by a comparable increase in positive moments. In 1985, AASHTO approved the use of a *Guide Specification for Alternate Load Factor Design Procedures for Steel Beam Bridges Using Braced Compact Sections*. This is commonly referred to as the Autostress Design Method and is another step towards making steel bridges more economical.

Autostress design can be considered an extension of the 10% redistribution allowed for load factor design. The method itself has been formulated over the last decade and started with a paper presented at the ASCE National Structural Engineering meeting in San Francisco, April 1973. The paper proposed the shakedown theory be considered as a practical method of plastic analysis for steel structures subjected to repeated loads. Classical shakedown analysis indicates permanent deflections eventually stabilize for loads below the shakedown limit. After several cycles of loading, the structure begins to behave elastically again. This comes about because of some local yielding of overstressed parts.

In continuous steel structures designed by Autostress, this local yielding occurs in the negative moment regions as soon as the structure is subjected to heavy loads. This automatically creates moments and forces which act as prestressing moments and forces, thus the term Autostress. At the present time, the method is only applicable to rolled beam bridges, but research is underway to extend the method to plate girders.

Autostress design's major advantage is in the simplification of continuous beams. Cover plates can be eliminated from the negative moment region, which reduces cost. It also does away with the cover plate termination detail which has been the source of fatigue problems on older bridges and additional costs on newer bridges. When applied to welded sections, it will result in fewer splices and thinner flange plates. The first structure built using Autostress Design methods was the Whitechuck River Bridge in the Mount Baker National Forest near Darrington, Wash. After the bridge was completed, the University of Washington conducted load tests in late 1982. The bridge performed as expected and showed no signs of distress during and after the tests (Fig. 1).

Fig. 3. Whitechuck River Bridge, a 1986 winner in AISC Bridge Awards competition.

New York State and Tennessee have recently let to contract structures designed by Autostress methods. Tennessee's structure is an 8-span continuous structure 872 ft long. It carries the Great River Road over the Obion River in West Tennessee near Dyersburg (Fig. 2). The spans were 100 ft-112 ft-112 ft-112 ft-112 ft-112 ft-112 ft-100 ft. The Autostress Method permitted use of W36 × 170 (A572) rolled beams full length without any cover plates. Total structural steel weight for the bridge was 956,153 lbs. and was bid at $.61/lb. The total cost of the girders was 17% less than the estimated cost of a comparable prestressed concrete bridge.

Since Autostress shifts moments from the negative moment region to the positive moment region, the weights were about the same as with regular Load Factor Design. Resulting deflections did not increase. Since it was possible to use non-cover plated rolled shapes in economical sizes, significant savings were achieved.

Bonners Ferry Bridge[2]

As previously indicated, the concrete industry entered the medium- to long-span bridge market by using prestressing. This principle is clear cut and well known. Normally, a compressive prestress is applied to the area where load would produce tension stresses. An amount of prestressing is applied so the section stays in compression, thereby eliminating the cracking associated with reinforced concrete.

However, prestressing is a procedure which can be applied to steel as well as concrete. Recently, a bridge proposed to cross the Kootenai River at Bonners Ferry, Idaho (Fig. 3) was designed initially using cast-in-place, post-tensioned concrete. In keeping with the general FHWA policy, an alternate bridge design was prepared us-

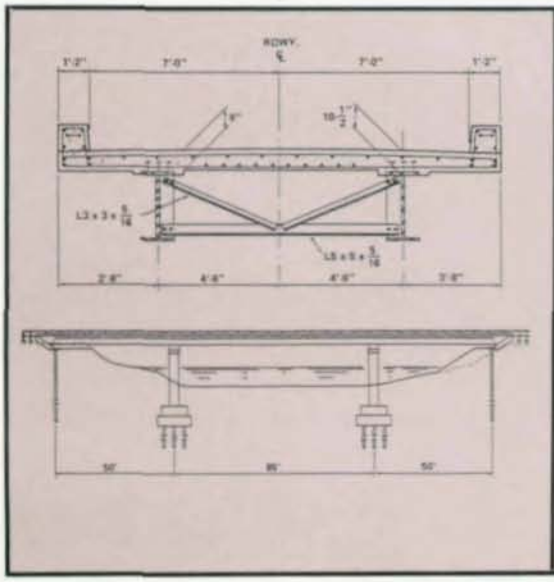

Fig. 1. Elev. of Whitechuck Bridge (top); cross section

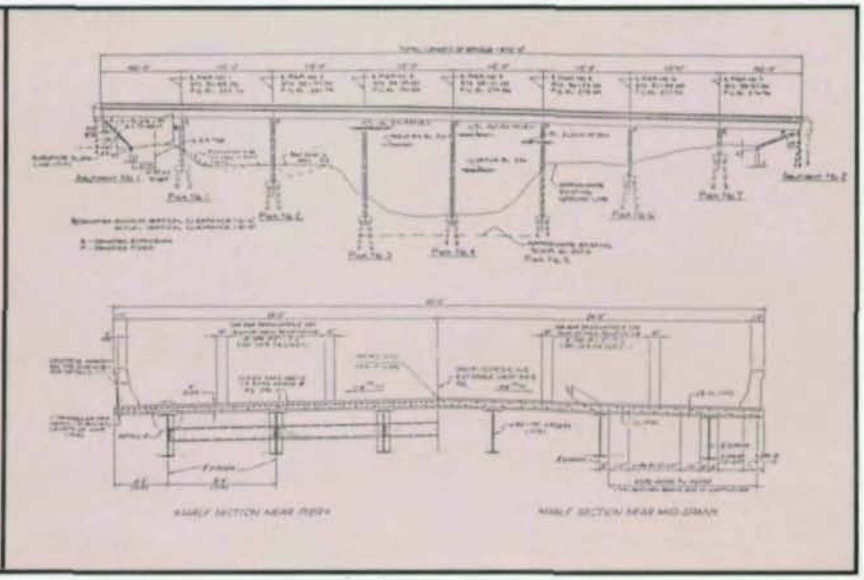

Fig. 2. Elev. (top) and cross section

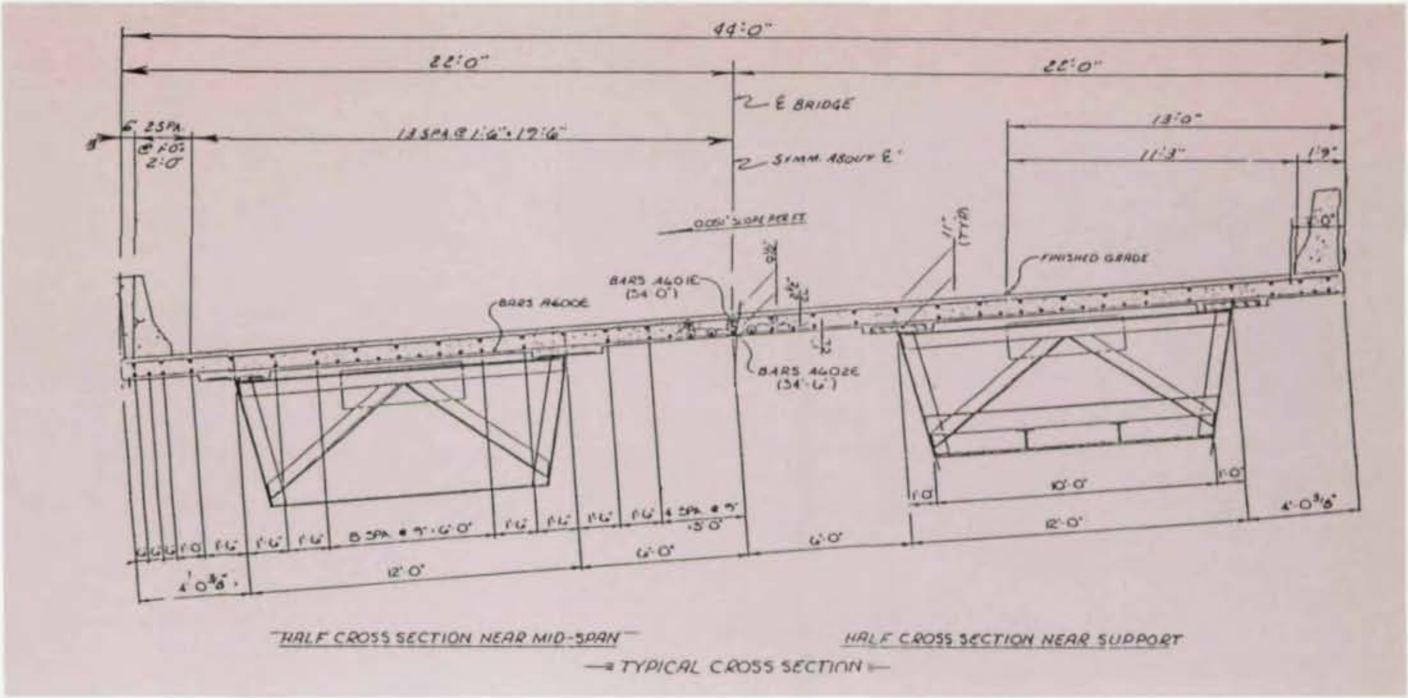

Figure 4

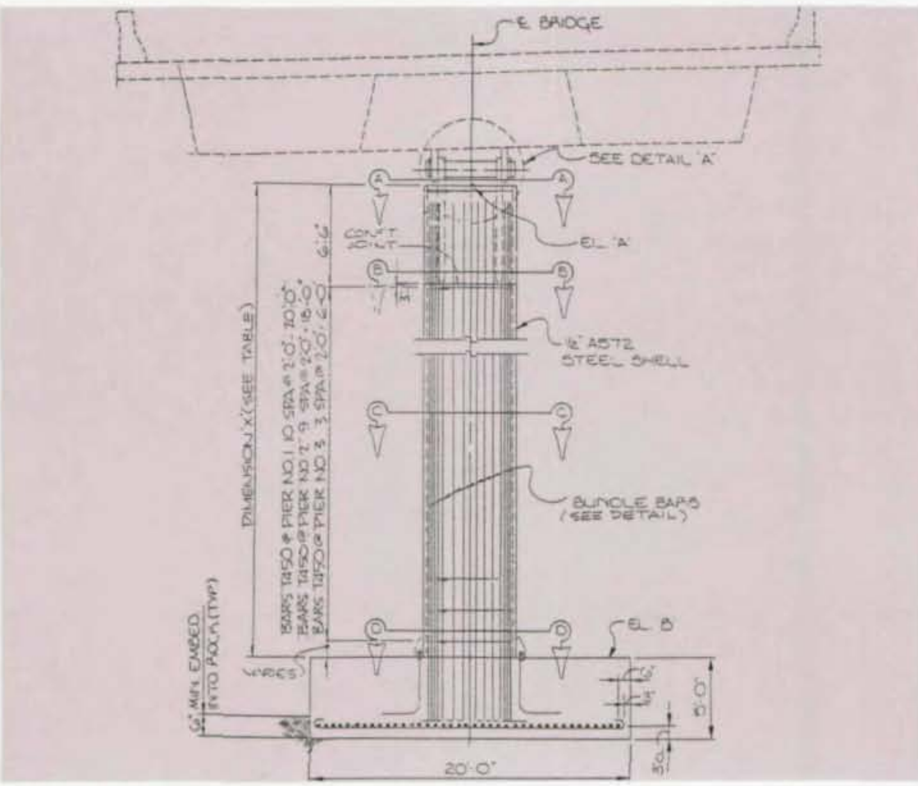

Fig. 5. Elevation

ing steel. T.Y. Lin International was employed by the Idaho DOT to study alternative steel structures that would be competitive with concrete. This study indicated innovative concepts using prestressing not only would be competitive but also preliminarily would be less expensive.

With this in mind, T.Y. Lin prepared plans for a steel structure that could be let in competition with concrete. Despite the innovative features of the design, the first steel bridge of its kind, the steel bridge was the low bid. In fact, all eight bidders chose the steel alternate.

The features included transverse prestressing of the deck slab and two-stage longitudinal prestressing of the deck slab and steel girders. By using shear connectors throughout and applying prestressing so the deck slab is always in compression, full composite action was achieved both in the negative moment regions over the pier as well as in the positive moment regions. This placed material where it was needed and reduced the weight of steel without sacrificing performance. The transverse prestressing allowed wide girder spacings with large overhangs. This reduced the number of girders, eliminated the need for stringers and thereby saved steel and fabrication costs.

Improved Lateral Live-load Distribution

Recently, many major bridge projects have employed techniques that yield more realistic live-load distribution factors. The AASHTO live-load distribution can be as much as 40% conservative. By using finite-element models, more exact distributive factors can be determined which reduces the weight of steel required. While this applies to concrete also, it has more effect on steel, since segmental concrete is often controlled by construction loadings.

The Melrose Interchange

On April 30, 1982 the Tennessee DOT let to contract one of the largest steel box-girder jobs in the U.S. in one four-level interchange with six bridges, a total of 4,427 ft in length (Fig. 4). A total of 3,800 tons of A36, 1,000 tons of A572 and 231,000 pounds of A588 were used. The longest bridge was 1,026 ft, with a maximum span of 281 ft. Roadways, which varied from 42 ft to 48 ft, were supported by single shaft, concrete-filled steel shell piers (Fig. 5). All bridges were continuous-curved trapezoidal steel boxes. Two significant details were used at the piers to simplify design and construction. To get around a biaxial

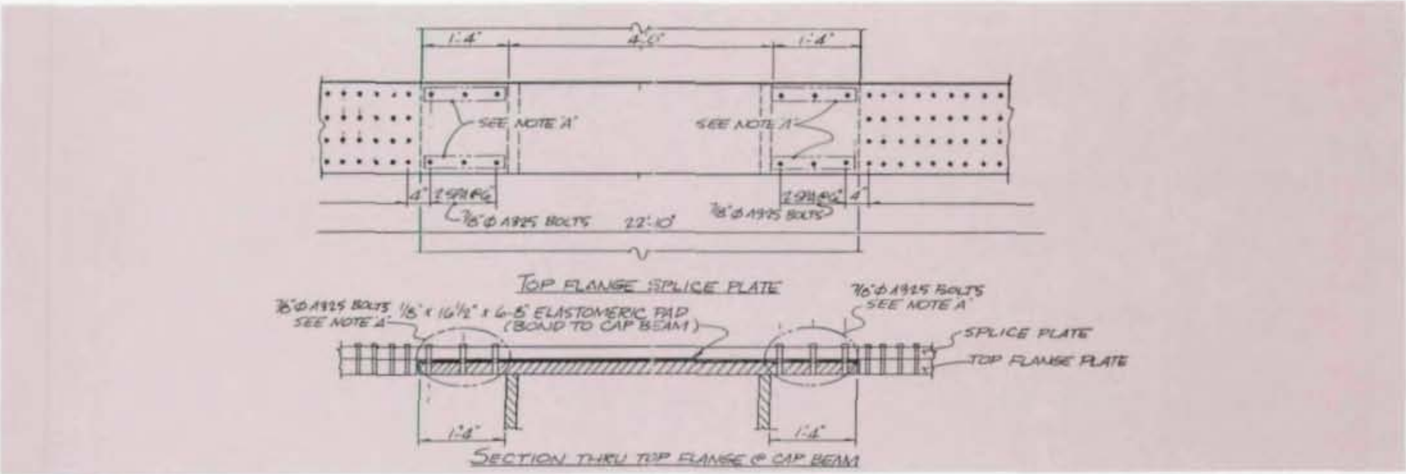

Figure 6

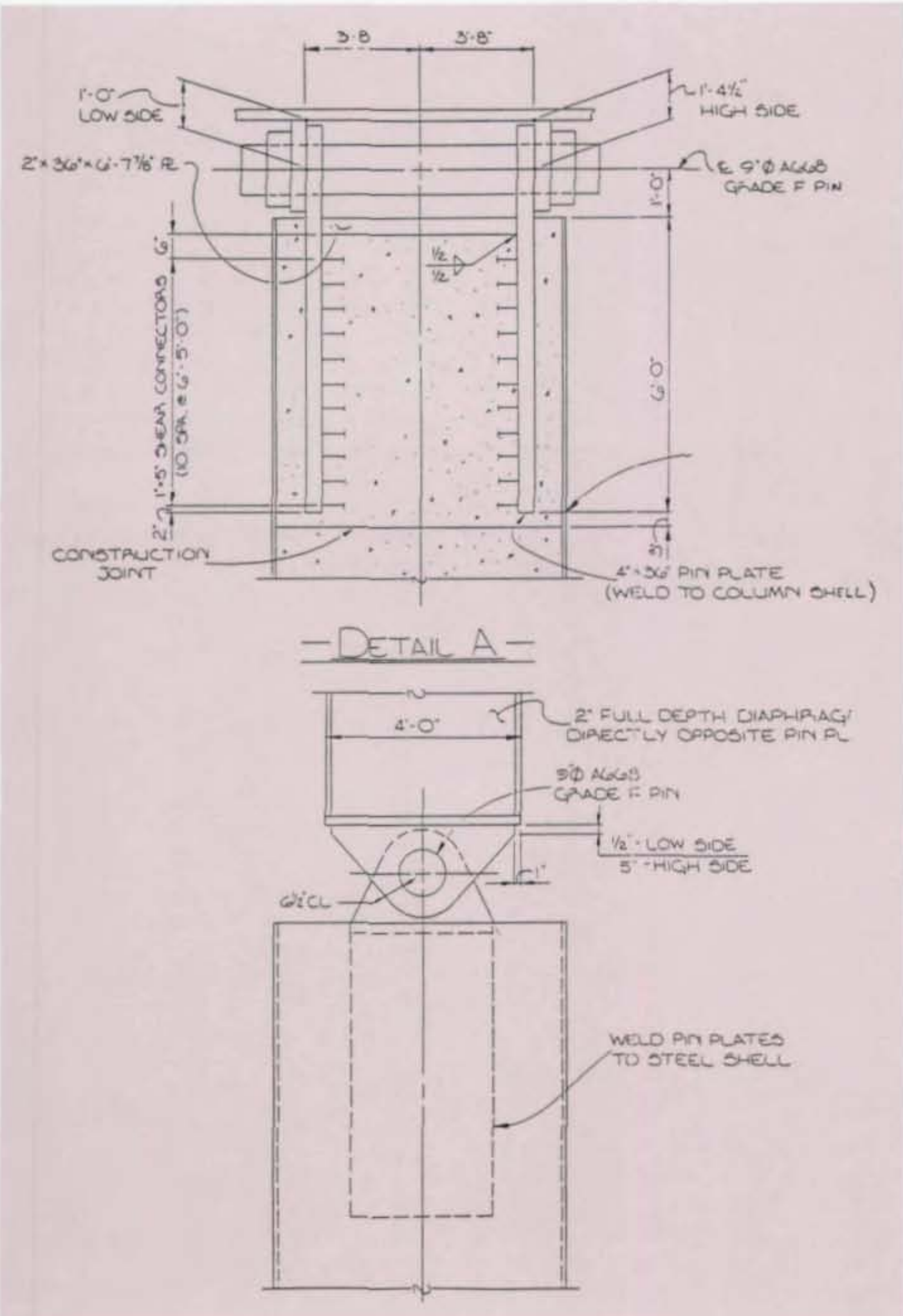

Figure 7

bending problem in the integral pier caps, designers used a detail as shown in Fig. 6. This allowed the forces to be transferred to the bent without creating biaxial bending. To provide for rotation and erection tolerances, they used a detail as shown in Fig. 7, which allowed erection to proceed smoothly.

Alternate Bridge Designs

As mentioned, the FHWA requirement on all major bridges is that an alternate must be considered—normally, steel against concrete. Having an alternate does not necessarily mean competitiveness. If the owner chooses, he can impose restrictions on one material or the other, thereby dictating which will win. In Tennessee, one reason steel has been competitive is the lack of restraint. We always allow the steel to be redesigned in alternate bidding with the only provision being that it is in accord with the AASHTO Standard Specifications for Highway Bridges. This permits innovation on the part of the contractor and fabricator, optimizing their best points.

In summary, steel has taken a few lumps recently. But it has demonstrated, that given a fair and equal opportunity, it can be very cost-effective. □

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Composite Systems

QUAKER TOWER

Chicago's Newest Waterfront

by Clark Baurer and John Zils

Soaring 35 stories above the Chicago River, the blue-green glass and polished-stainless, steel-clad, 1,050,000-sq. ft Quaker Tower is the first phase of the Riverfront Park Development. On the north edge of the Chicago River, Riverfront Park will include a hotel, Phase IB and a connecting link between the Tower and the hotel. All are now under construction, with two additional office buildings north of Phase I planned for the future. The strategic siting of Quaker Tower grants it high visibility as well, and, affords tenants a panoramic view of both Loop and lakefront.

As its name indicates, Quaker Tower is the new international headquarters for Quaker Oats, who will occupy about half the building. In addition to their office floors, Quaker will have a private cafeteria and conference center in one of the four *levels below the lobby. The cafeteria and* conference center are adjacent to a private outdoor terrace along the river edge. Also at river's edge, one level below Quaker's outdoor terrace, a brazier restaurant opening onto a public riverfront terrace will be accessible from the development's landscaped walkway.

The lobby area will contain the main reception area for Quaker Oats at the south end and a banking facility at the north end. The central elevator core is clad in marble, and arched openings leading to the elevator lobbies are sheathed in travertine. The exterior of the 25-ft high lobby is full height, clear glass, to give the lobby an expansive, open, spacious, light-filled ambience.

Structural System

In keeping with the concept of an open and light-filled lobby and office space to optimize views to the outside, it was essential the structural system at the perimeter of the building be as light as possible. This

was accomplished by providing the lateral wind-resistant system entirely within the central core area of the building. Several structural systems were evaluated and the one chosen, for the reasons already noted, as well as for economic reasons, was a composite system comprised of a concrete interior shear wall forming the central core, exterior steel gravity columns and simple composite, steel floor framing.

Lateral System

Lateral stability for the building is provided by a reinforced concrete shear wall which forms the central building core. The shear wall is cantilevered from a mat foundation which rests on reinforced concrete belled caissons that extend to hardpan strata 70 ft below river level.

The elevators have three banks—low-rise, mid-rise and high-rise. At the termination of the low- and mid-rise elevator banks, the building core size is reduced. Since lateral stiffness requirements reduce higher in the building, the concrete shear walls can be dropped off or stepped back at the termination of the low- and mid-rise elevator banks. The lateral loads are transferred to the shear wall from the exterior of the building through the rigid composite metal deck and lightweight concrete floor slab diaphragm. The floor slab is engaged to the concrete core with reinforcement anchored in coil loop inserts set in a keyway at the perimeter of the shear wall.

The three bay frames at the north and south ends of the building were moment connected to provide additional torsional stiffness.

Gravity Framing

With the lateral system established and located at the interior of the building, floor framing could be designed to optimize cost, floor-to-floor height and lease span requirements. The core size and buidling dimensions were previously determined, resulting in a lease span of 45 ft from core to exterior wall. Since it was essential this space be column-free, the floor framing system was required to span this distance. A series of studies determined the most economical system. The lightest floor framing member for this span was determined to be a built-up steel truss. Trusses have higher fabrication costs. However, trusses were considered in this project because the regular shape of the floor plan permitted a great deal of repetition of truss members, thereby simplifying fabrication.

In addition to economic considerations, the trusses have other advantages:

1. Since mechanical systems can pass through the truss, a greater part of the

ceiling sandwich depth can be used for structural framing without increasing floor-to-floor height. Furthermore, trusses may be designed to accommodate specific opening sizes and locations, giving them flexibility and adaptability to various mechanical systems.

2. Since the truss uses the entire depth of the ceiling sandwich, it forms a very stiff floor system, thereby minimizing deflection and floor vibrations.

The final design for this project was a system of 37-in. deep, built-up, composite steel trusses (A36, 36 ksi), spaced 15 ft o.c. spanning from exterior gravity columns (A572, 50 ksi) and spandrels (A36, 36 ksi) to corbels at the concrete shear wall (see Figs. 1 and 2). At the north and south ends of the building, the trusses span from exterior gravity columns and spandrels to 37-in. deep, built-up composite truss girders (A36, 36 ksi). A 3-in. composite metal deck slab with 2½ in. of lightweight concrete spans the 15 ft between floor trusses. Welded steel studs on the top flange of trusses transfer shear between slab and truss to provide the required composite action.

The floor truss girders and trusses at plaza and mechanical floors are built-up of WT top and bottom chords and double-angle diagonals. For the typical office floor, trusses were built-up of single-angle top and bottom chords and single-angle diagonals. The single-angle trusses are slightly heavier than a truss of the same span built-up of WT's and double angles because of the inherent eccentricity in the connection of two single angles. However, since angle sections are less expensive and more readily available than WT sections, this weight premium was offset, resulting in a more economical system. An additional, potential economy in the single-angle trusses compared to trusses built-up of WT's and double angles is in fabrication. Since all the web members are applied to one side of the chord, it is possible to weld the entire truss without turning the truss over, thus saving shop fabrication time.

The trusses proved to be the most economical system since the savings in tonnage, compared to a wide-flange system (approximately ½ psf), offset the higher cost of fabrication. The floor trusses were customized to integrate the mechanical system used in the project. The mechanical system required a continuous supply air duct approximately 28 in. wide and 16 in. deep to circle the floor at midspan of the floor framing. Since a duct of this size could not be accommodated in the triangular openings between truss diagonals, a rectangular opening was provided at mid-

Fig. 2. Floor trusses

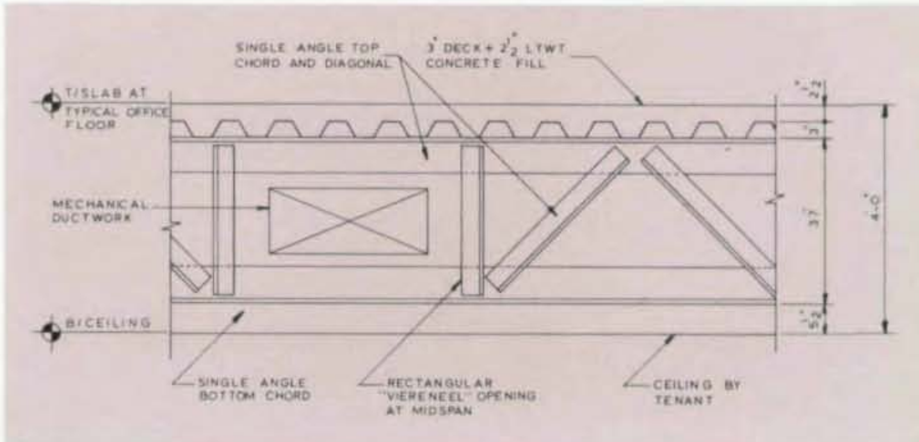

Fig. 3. Ceiling sandwich detail at truss

span of the truss by orienting two web members vertically (see Figs. 3 and 4). This rectangular opening was possible in the truss member because it was located at midspan where shear is low. The small amount of shear present was carried by Vierendeel action, employing the stiffness of the web members and the fixity of the welded connections of the web members to the top and bottom chord. This customization allowed the mechanical and structural systems to be integrated within a 4-ft ceiling sandwich, so an 8 ft-6 in. ceiling height could be provided using a 12 ft-6 in. floor-to-floor height.

In addition, the trusses provided greater stiffness than a wide-flange framing system. The floor trusses used were more than six times stiffer than the required wide-flange beam, yet ½ psf lighter. This demonstrates a very efficient use of material.

Differential Shortening

As on any high-rise construction, the unpredictability of loading patterns and temperature changes combine to make differential shortening of vertical elements difficult to predict. This problem is complicated further by the use of two different materials, such as a concrete shear wall and steel gravity columns.

A preliminary study addressed this problem. A specific construction sequence was provided by the general contractor which formed its basis. Building dead loads were determined and construction live loads estimated. Given the strength, stress and volume-to-surface ratio of the concrete, certain predictions were made regarding the anticipated elastic shortening, creep and shrinkage of the shear core. A computer model was built according to the given construction sequence, floor by floor through the completion and occupancy of the building, to study the cumulative elastic shortening of the steel columns, the cumulative elastic shortening, creep and shrinkage shortening of the concrete—and how the two related.

From this study came the conclusion the concrete would cumulatively shorten more than the steel, and that to achieve level floors at the completion of the building, the concrete core would need to be over-lengthened. Given this, a computer program was developed in which the contractor would provide the engineer information as to the completion of certain critical items (pouring of deck slab of a certain floor, for example), then the computer model would be updated, given the amount of load on the structure at that point in time. A shear wall overlength would then be given to the contractor, and he would establish the benchmark for that particular floor at the theoretical elevation plus the suggested overlength.

This procedure was carried out. The only test thus far of the process was at the 16th floor, which was predetermined as a leveling off floor. At the 16th floor, the actual benchmark on the concrete core was compared to the top of steel elevation of exterior columns at the core. The results of this survey compared favorably with the predicted behavior.

Conclusion

By providing the lateral stability entirely within the central building core, thereby minimizing the size of structure at the exterior, the design intentions of a light-filled, expansive lobby with optimal views to the outside were realized. By using built-up steel floor trusses, the 45-ft clear lease span was possible. Because of the integration of the mechanical system with the floor trusses, this clear span was provided while maintaining a standard floor-to-floor height. And the efficient use of structural steel provided this span with a comparatively light, and very stiff, floor system.

Quaker Tower forms a dramatic first installment in Riverfront Park, a very exciting and dynamic North Loop development. In addition to the high quality of the development itself, the project provides a significant link across the river, connecting the Loop with the North Loop development. With public terraces at river level, an open and spacious lobby, and office space with clear lease spans and panoramic views of the river and Loop, Quaker Tower signifies a renewed interest in developing the North Loop and the river as an amenity for Chicago. □

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Quaker Tower, Chicago's newest "waterfront." Dockside amenities, below. Truss girders (l.) frame well-known Marina City.

Steel Notes

STEEL BRIDGE SYMPOSIUM SET FOR SEPTEMBER

The National Symposium on Steel Bridge Construction—co-sponsored by AISC, the Federal Highway Administration (FHWA) and the American Association of State Highway and Transportation Officials (AASHTO)—will be held Sept. 14 & 15, 1987 at *The Shoreham Hotel*, Washington, D.C. The day-and-a-half program offers eminently qualified speakers presenting current practices and directions for future advances in steel bridge construction. The Symposium's theme is: "To create a dialogue between owners, designers and builders to enhance the economy, quality and reliability of steel bridges."

The first day of the Symposium, Sept. 14, offers presentations on quality assurance and control, use of weathering steel and standardized bridge details. Following the evening banquet, Ray H. Barnhart, administrator of FHWA, will address attendees. Autostress application, fracture critical members, erection considerations and how to get the project built are the topics for the Sept. 15 presentations.

The Symposium was designed to meet the need for a first-class, hands-on national symposium dealing with the specifics of bridge construction. Fabricators, erectors, designers, owners and bridge constructors will benefit by attending this premier event.

Many FHWA, AASHTO, State DOT representatives and consulting engineers involved in the development of the program will be present.

For more information, contact AISC Research/Engineering Department, 400 N. Michigan Ave., Chicago, Ill. 60611-4185; 312/670-5413.

ATLSS SPONSORSHIP INVITED

Lehigh University, the designated National Engineering Research Center on Advanced Technology of Large Structural Systems (ATLSS), is looking for financial support and membership for the program.

The center, under the direction of Dr. John Fisher, concentrates on various topics relevant to steel buildings and bridges, including connection design and construction. It is expected the technological advances developed will help reduce costs and make steel a more competitive construction material.

For more information on the center, or how to participate in the program, contact Ruth Grimes, ATLSS, Fritz Engineering Laboratory, Building 13, Lehigh University, Bethlehem, Pa. 18015; or call 215/758-3535.

PRELIMINARY WORK STARTED ON REVISED ALLOWABLE STRESS DESIGN SPECIFICATION

The AISC Committee on Specifications is considering several revision proposals for updating the Allowable Stress Design (ASD) Specification. Although the AISC Long Range Strategic Plan calls for maximum utilization of Load and Resistance Factor Design (LRFD), continued reliability of ASD must also be ensured.

LRFD LECTURES CONTINUE TO DRAW

In the first four months of 1987, the Load and Resistance Factor Design (LRFD) lecture program visited 26 cities with a total attendance of over 4,500 design professionals. The five-lecture educational package covers the essential procedures in the LRFD Manual, which introduces more uniform reliability in structural steel design.

Upcoming lectures scheduled are: May 7 in Phoenix, Ariz.; May 12 & 13 in Troy/Albany, N.Y. and in Irvine, Cal.; May 20 in Syracuse, N.Y.; May 20 & 21 in Kansas City, Mo; June 2 & 3 in Miami; June 17 & 18 in San Antonio, Tex.; and June 30 & July 1 in Birmingham, Ala.

Twenty-eight additional cities across the U.S. are targeted for programs. Information will be sent to local design professionals once a program date and location are scheduled. For further information, call Janet Manning, AISC headquarters, 312/670-5431.

LRFD CODE STATUS

As the new AISC Load and Resistance Factor Design (LRFD) Specification has become available, questions have been raised about its acceptance by the model and local building codes.

AISC is recommending that the appropriate code change language recognizes LRFD as an *alternate* steel building design method to the current AISC Allowable Stress Design rules. Such provisions are already contained in the 1987 BOCA National Building Code and have been approved in principle by the Southern Building Code (SBCCI) and Uniform Building Code (ICBO), but due to procedural delays, will not be formally included in these latter two model codes until 1988 or so. Hence, there may be a lag time before some local building authorities take formal action in implementing these changes.

Any questions on the subject? Contact David Jeanes, AISI, 202/452-7178 or Nestor Iwankiw, AISC, 312/670-5415.