VOLUME IX / NUMBER 2 / SECOND QUARTER 1969

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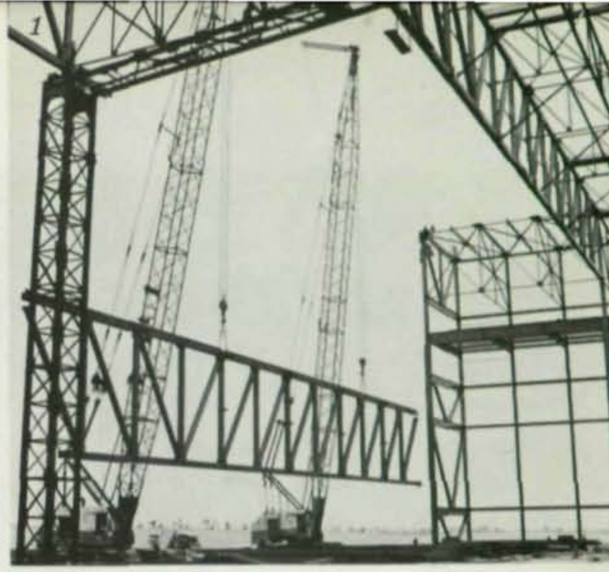
MODERN STEEL CONSTRUCTION



1 Steel in the Desert . . . page 8
2 Balanced Design for Apartment Tower . . . page 14
3 Lightweight Dome at Ohio University . . . . . page 3

Steel

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CONTENTS

Lightweight Dome at Ohio University 3
A Floating Steel Fountain 6
Steel in the Desert 8
Lifting the Glass at Brandywine 11
Balanced Design for Apartment Tower 14

1969 ARCHITECTURAL AWARDS OF EXCELLENCE COMPETITION

The American Institute of Steel Construction has announced the opening of its tenth annual Architectural Awards of Excellence Program to recognize and encourage creative uses of steel construction. The 1969 Awards will salute outstanding aesthetic design with structural steel.

All registered architects practicing professionally in the United States are invited to enter steel-framed buildings of their design constructed anywhere in the 50 states and completed after January 1, 1968 and prior to September 1, 1969.

The structural frame of the building must be steel, although it is not a requirement that the steel be exposed and a part of the architectural expression. Buildings of all classifications are eligible, with equal emphasis given to all sizes and types in the judging. There is no limit to the number of entries by any individual or firm.

The competition will be judged by a panel of five distinguished architects and engineers. The members of the 1969 National Jury of Awards are:

Rex Whitaker Allen, FAIA *President-elect American Institute of Architects; Rex Allen & Associates, San Francisco, California*

Jacques C. Brownson, AIA *Managing Architect, Public Building Commission of Chicago, Chicago, Illinois*

John Dinkeloo, AIA *Kevin Roche John Dinkeloo & Associates, Hamden, Connecticut*

Dr. James M. Paulson, M.ASCE *Chairman, Department of Civil Engineering, Wayne State University, Detroit, Michigan*

Walter F. Wagner, Jr., AIA *Editor, Architectural Record, New York, New York*

Competition rules are available from AISC, 101 Park Avenue, New York, New York 10017. Entries must be postmarked prior to September 1, 1969.

Lightweight Dome at Ohio University

by Russell S. Fling

One of the lightest Schwedler domes in the United States crowns the multi-purpose Convocation Center at Ohio University in Athens. The Center houses 360 students as well as a basketball arena and seating for over 13,000 spectators and related facilities for the physical education department. The dormitories are tucked beneath the bleachers and have a complete separation from the arena and physical education facilities, including separate stairs and entrances.

The arena contains loud speakers, a T.V. booth, portable stage, and other facilities for convocations, shows, as well as for sporting events. Locker rooms, handball courts, athletic offices, and ancillary facilities are located beneath the dormitories and bleachers.

The Center is in a flood plain, so the entire structure and approach ramps were raised 10½ ft above the surrounding ground level to place it 3 ft above the highest known flood water level. The resulting space beneath the Center is used to park 250 cars.

Steel Domes

The 328-ft diameter steel Schwedler dome roof is supported on 48 concrete columns. Twenty-four 24WF steel ribs radiate from a 34-ft diameter compression ring. Three hoop beams divide each pie shaped section into 4 facets supporting 24-in. deep short span steel joists. Thus the dome is like a finely cut diamond with 96 trapezoidal facets and one round center facet.

Thrust of the dome is taken by a spider web of steel cables extending horizontally across the dome at the spring line, assisted by a concrete ring girder.

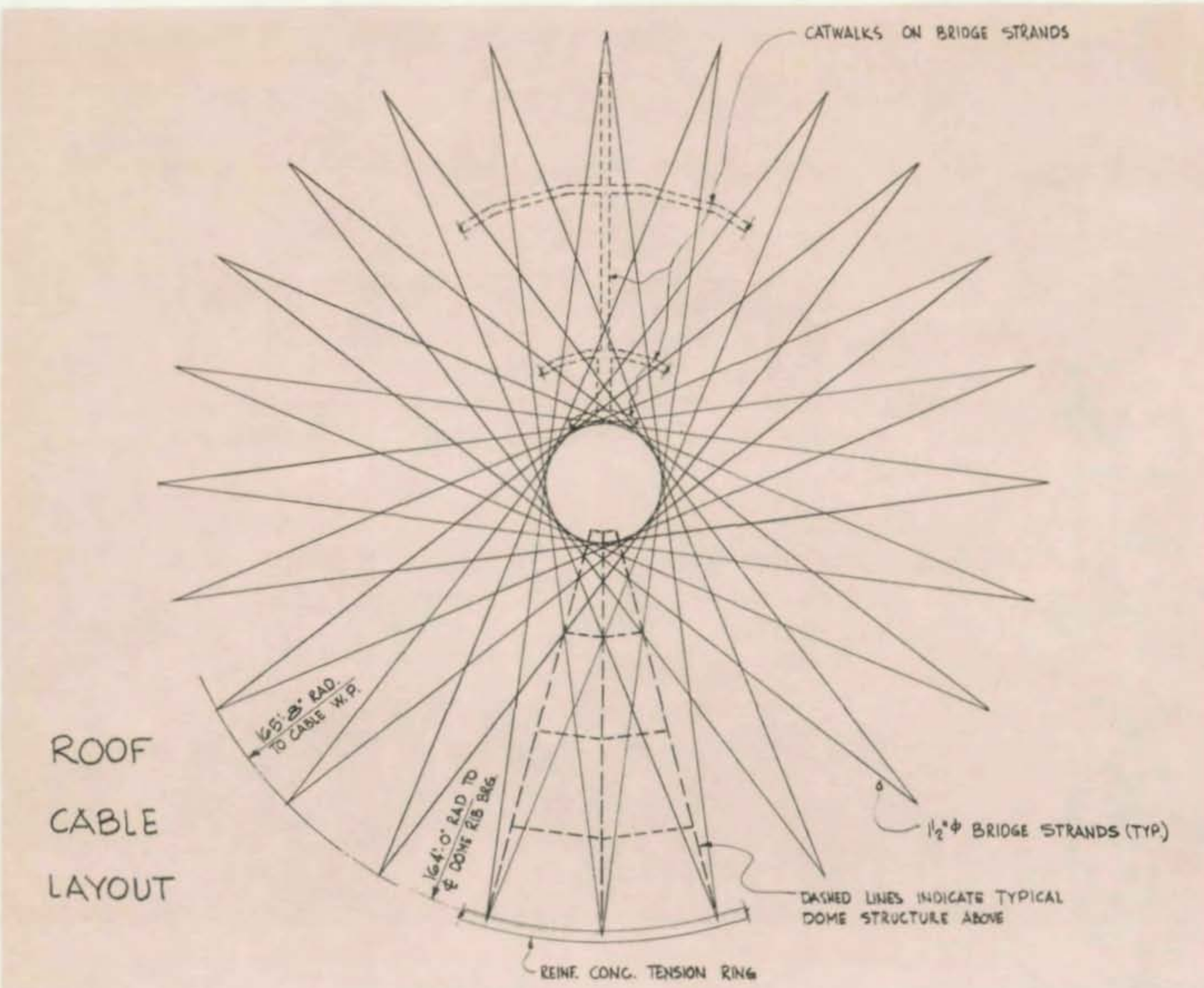
Architect:
Brubaker & Brandt
Columbus, Ohio

Structural Engineer:
Fling & Eeman, Inc.
Columbus, Ohio

General Contractor:
Knowlton Construction Co.
Bellefountaine, Ohio

Steel Fabricator:
Bristol Steel & Iron Works, Inc.
Bristol, Va.

Russell S. Fling is a principal in the firm of Fling & Eeman, Inc., Consulting Engineers, Columbus, Ohio.



ROOF CABLE LAYOUT

The ribs are braced by diagonal angles running to the intersection of the ribs and hoop beams in four bays. In addition, the normal joist bridging and bulb tees for the roof deck are utilized to provide further bracing for the ribs between hoop beams.

The dome was analyzed as a three-dimensional framework using conventional methods of stress analysis. Each hoop beam was analyzed for three loading conditions, namely: full live load over the entire dome, live load above the hoop and no live load below, live load below the hoop and no live load above. The most critical of the three conditions was used for design.

To analyze stresses in the bridge cables as well as to study the sequence of erection, a model was constructed using string connected to small springs at each column point in the prototype. A horizontal load applied at any one column is reflected in diagonal forces at all other column points, varying in magnitude with their proximity to the loaded column. From this, it was obvious that a ring girder is essential in addition to the cables and that the cables could exert small lateral loads on the dome ribs under unbalanced loading. These lateral loads presented no problem to the final design of the dome.

Other Features

Three annular rings of catwalks are supported directly on the steel cables and lighting for the arena is attached to the catwalks. Thus both installation and maintenance of the general lighting system was greatly facilitated. Cost of the lighting was also minimized by lowering its height from the underside of the roof. Walking on the catwalks creates a gentle undulation not unlike a sailing yacht with a similar feeling of confidence in the ultimate strength and stability of footing.

Since domical surfaces tend to focus reflected sound on the source, a wood fiber roof deck was used. This type of deck has a high sound absorptive capacity through the full range normal frequencies.

The 24-in. x 48-in. columns are jointed at 10½ ft below their tops to permit expansion and contraction of the tension ring due to temperature and live loading. Every other column, supporting a dome rib, has a joint composed of 3-in. thick laminated neoprene pads. Intermediate columns are supported at the joint with steel rockers which permit rotational

movement about a tangential axis only. The rockers were set on ¼-in. shims which were later removed to equal the compressive strain in the neoprene pads.

The entire structure is supported on straight shafted 18- to 42-in. diameter caissons drilled 50 ft to solid bedrock to reach 40 tons per sq ft bearing capacity. Caissons under exterior columns are topped with an 11-ft long x 3-ft high cap to transmit horizontal wind forces into the ground.

Erection

Erection of the dome proceeded in a normal manner, utilizing a square support tower at the center with 12-in. diameter pipe shores under each of the 24 ribs near their mid-point. All four tower legs and each shore had a screw jack for adjustment to final grade. The contractor originally contemplated using only 13 shores by moving them one at a time after the first half of the dome had been erected. However, it soon became evident that the 3-ft high x 4-ft wide concrete tension ring, massive as it is, could be easily distorted by the weight and arching action of a single pair of opposing ribs.

When erection was nearly complete, one shore fell after a particularly cold night. At first it was thought the cold temperature induced a brittle fracture of the screw jack. Later analysis indicated the more probable reason was a greater thermal shortening of the steel ribs than of the concrete tension ring. Shortening of the ribs lowered the dome and overloaded the shores. This overstress caused one of the jacks to fail and to allow collapse of the shore.

Minimum Steel Weight

The steel dome as constructed uses an amazingly low 8.9 lbs of steel framing per sq ft of vertical projection. The steel cable and reinforcing steel in the ring girder amounted to an additional 1.4 lbs per sq ft. Most domes of a similar span have used considerably more steel. The steel is distributed as follows:

Structural Steel	230 tons	5.5 psf
Steel Joist	115	2.7
Bracing and Joist Bridging	31	0.7
Total Steel Framing	376 tons	8.9 psf
Bridge Strand	22	0.5
Reinforcing Steel in Concrete Ring	37	0.9
TOTAL	435 tons	10.3 psf

Lightweight Dome (cont'd)

This extremely low weight is achieved in five ways:

1. The number of ribs is held to a minimum so that each rib is fully stressed. This factor is especially important near the center of the dome where an excessive number of ribs can result in an inefficient use of closely spaced steel members.

2. Hoop beams are designed for continuity, which considerably reduces the magnitude of the bending moment. Connections are simple end plates bearing on the web of the ribs and generally require no further provision for continuity because the extreme fiber stress due to bending is less than that due to axial compression.

3. Steel joists are used extensively to support the roof deck. Steel joists are well known for their extreme light weight and efficient use of material. In addition, the outer ring of joists deliver half their load directly to the supporting concrete structure and thus relieve the dome of approximately 22% of its roof load.

4. The dome thrust is resisted by high strength cables and by high strength reinforcing steel, both of which have a low cost/strength ratio. Cost of the bridge cables is minimized by omitting the usual center tension ring. This cuts the number of end connections in half and reduces the cost of the cables by approximately ¼ as well as saving the entire cost of the tension ring. The resulting pattern of cables creates a lacy spider web decoration to the arena as well as providing a 43-ft diameter opening in the center of the arena. (See figure.) This opening allows an extensive system of sound amplification equipment to be installed at the center of the arena. The efficiency of the bridge cables is further increased by using them to support the catwalks and lighting as described earlier.

5. The faceted surface of the dome permits the use of straight members. Thus standard steel joists can be used and the fabrication cost of the structural steel can be reduced.

Not reflected in the weight or cost of the dome are the further economies realized in the roof decking and aluminum roofing due to flat surfaces.

A permanent, floating steel fountai[n] which throws a jet of water 250 ft int[o] the air (nearly half the height of th[e] Washington Monument), marks one o[f] Mrs. Lyndon Johnson's final beautifi[-] cation efforts in the capital as the na[-] tion's First Lady. Floating in the Poto[-] mac River, the illuminated fountain's je[t] spray can be seen for miles on bot[h] sides of the river from March 1 to No[-] vember 1 at regularly scheduled times.

Late last year the fountain was an[-] chored about 200 ft from the seawall of Hains Point, at the southern tip of Eas[t] Potomac Park. Anchorage is provided b[y] four one-cubic-yard concrete deadmen. Chains 38 ft long fasten the fountain t[o] the deadmen and allow it to rise and fal[l] with varying water levels. There is 7 f[t] of water at the site. The fountain ca[n] be brought ashore for maintenance.

Steel Construction

Although the steel fountain is 5 f[t] high, little of it can be seen. Only 18-in of it is above water, partially obscured by the jets and a fiber-glass shield.

The perimeter structure forms a dode[-] cagon, 33 ft in diameter, and is con[-] structed of 12 watertight tanks fabri[-] cated from 3/16-in. plates of A36 steel. Each tank is 5 ft deep, with an outside width of 8 ft-6½ in. and an inner width 6 ft-9 in. The top of each tank contains a watertight 12-in. inspection port.

Three tanks were welded together t[o] make a subassembly. Four such subas[-] semblies were bolted together at the jobsite.

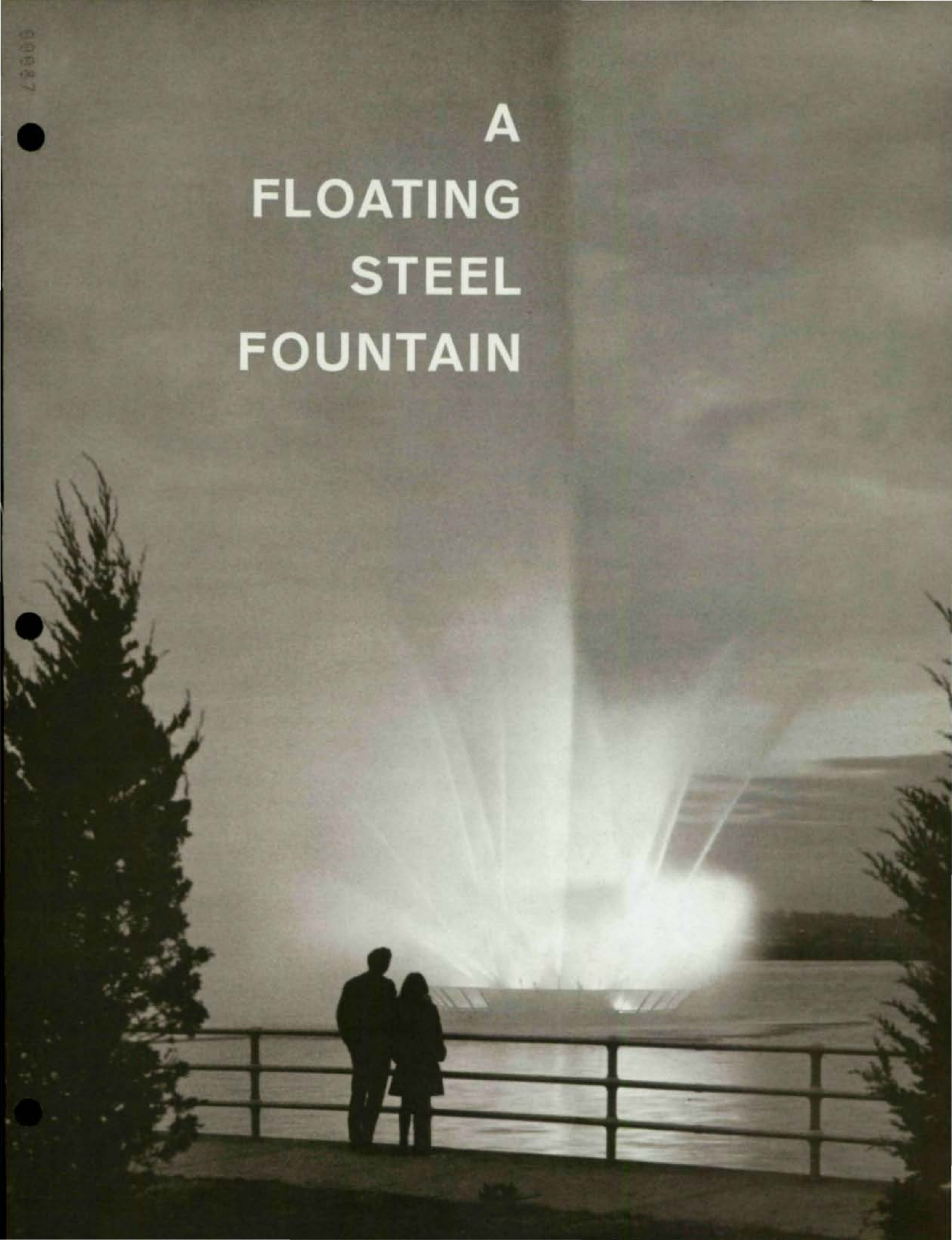
Support and rigidity were provided b[y] using 5/16-in. A36 stiffeners in the to[p] and bottom frames. The top frame als[o] supports the steel grating, lights, and the nozzle for the main jet. All fasteners were A325 steel.

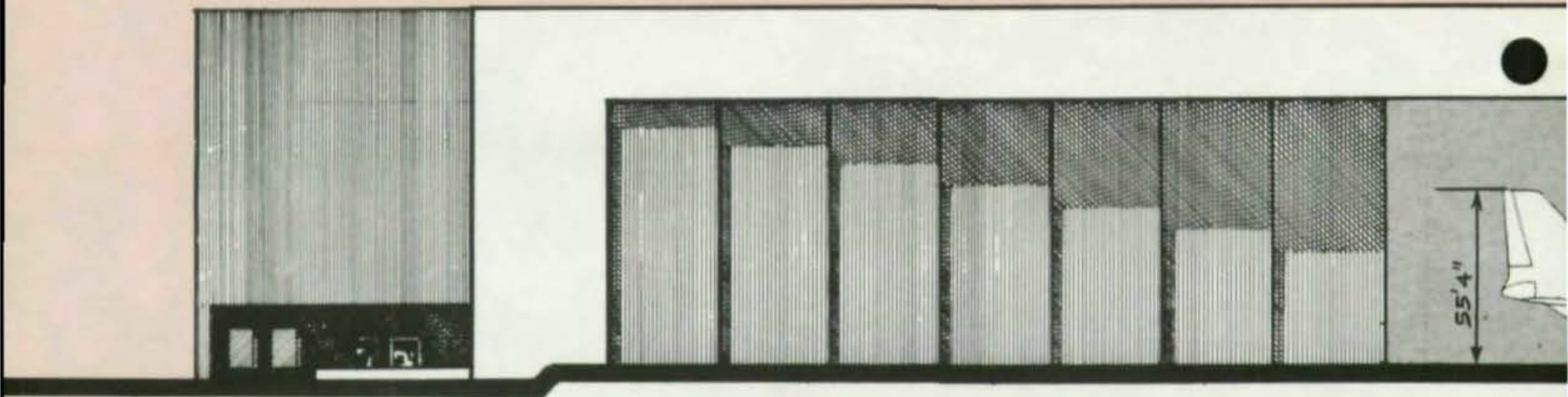
The pumps were bolted to cradles which were in turn welded to the tanks between the upper and lower framing.

All steel was sandblasted to white metal in the shop, and then a primer and two coats of epoxy paint were applied t[o] the steel. Touch-up work was finished after field erection.

Nelson, Dollar and Blitz, Washington, D. C., coordinated the project as archi[-] tect-engineer for Designer J. S. Hame[l] Engineer, Burbank, California. Structural engineer was Heinzman and Clifton, Washington, D. C.

A FLOATING STEEL FOUNTAIN





STEEL IN T

Among the ten structures comprising the Lockheed California Company's "TriStar" aircraft complex, currently under construction on the Mojave Desert at Palmdale, California, is the 871,500 sq ft Structures Mate and Final Assembly plant.

According to the architect, William L. Pereira & Associates, the "basic challenges in the design of this facility were its enormous size and large floor area spans, learning the assembly processes of aircraft in order to provide the necessary utility requirements at various points of assembly, and accomplishing this in a very short period of time."

The Structures Mate and Final Assembly plant occupies 180 acres of the entire complex's 539-acre site. The perimeter of the building is 900 ft x 640 ft, enclosing a 210-ft x 900-ft mating and assembly area, and a 350-ft x 900-ft final assembly area. The height of the building from the finish floor to the top of the parapet wall is 118 ft with an 85-ft clear height in the assembly bay. The structure's design has been kept as simple as possible due to its immense size.

TriStar's Vital Statistics

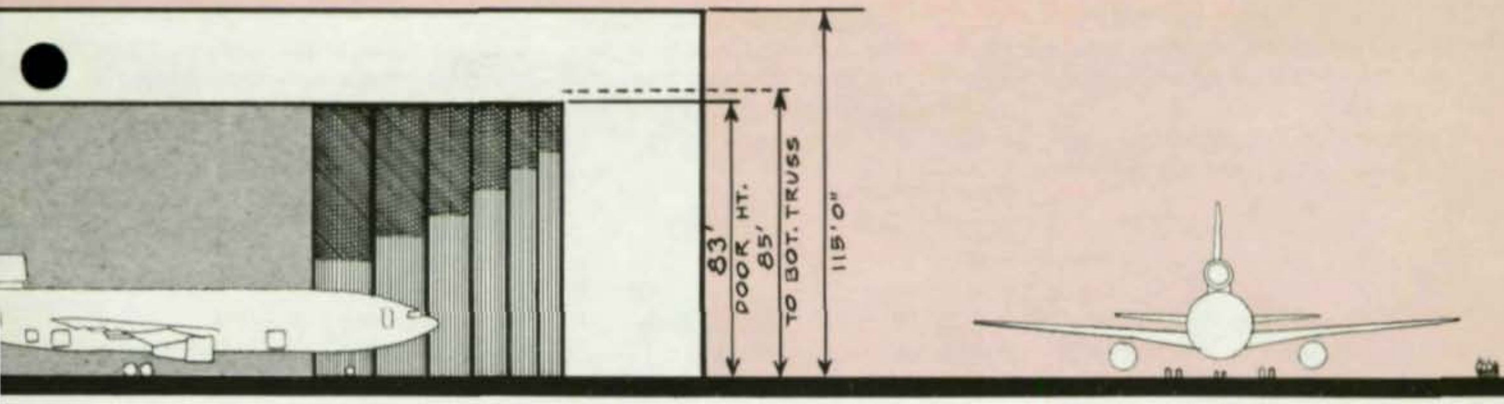
The size of the building was determined in part by the dimensions of the TriStar aircraft. TriStar (formerly designated the L 1011) has a wing span of 155.3 ft. Maximum length is 177.7 ft and maximum height is 55.3 ft. The plane's empty weight totals 225,000 lbs.

The aircraft will carry 245 first class passengers and 356 coach travellers. Cargo capacity weight will be 83,000 lbs. TriStar will feature 3 RB-201 Rolls Royce jet engines, each capable of generating a 40,000 lb thrust, approximately twice the thrust of current engines.

Structure Presents Engineering Challenge

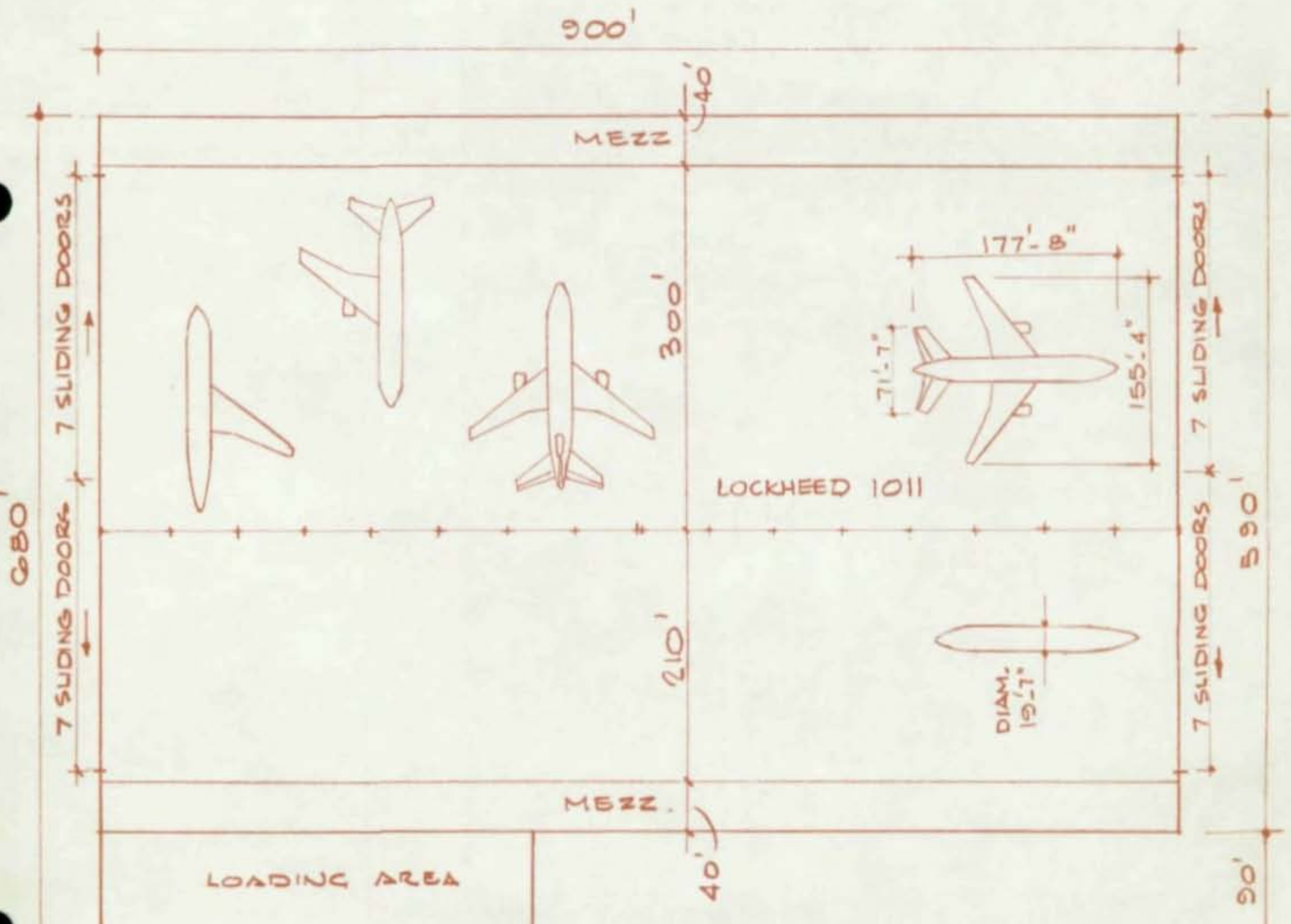
The most difficult problem for the structural engineers was to enclose the huge volume with minimum space for lateral bracing. The bracing system had to be rigid enough to resist seismic and wind loads, yet flexible enough to allow movement due to temperature change.

The main roof trusses span 210 ft and 300 ft across the building. In the 210-ft



83'
DOOR HT.
85'
TO BOT. TRUSS
115'0"

HE DESERT



900'
40'
MEZZ
7 SLIDING DOORS
300'
177'-8"
71'-7"
155'-4"
7 SLIDING DOORS
LOCKHEED 1011
680'
7 SLIDING DOORS
210'
DIAM. 19'-7"
7 SLIDING DOORS
590'
MEZZ.
40'
LOADING AREA
90'
360'

x 900-ft area, the trusses are 24 ft deep. In the 300-ft x 900-ft area, they are 27 ft deep. Steel purlins frame into the trusses at 10-ft-0 in. o.c. and carry steel roof decking. Top and bottom chords of the trusses are high strength A572-42 steel.

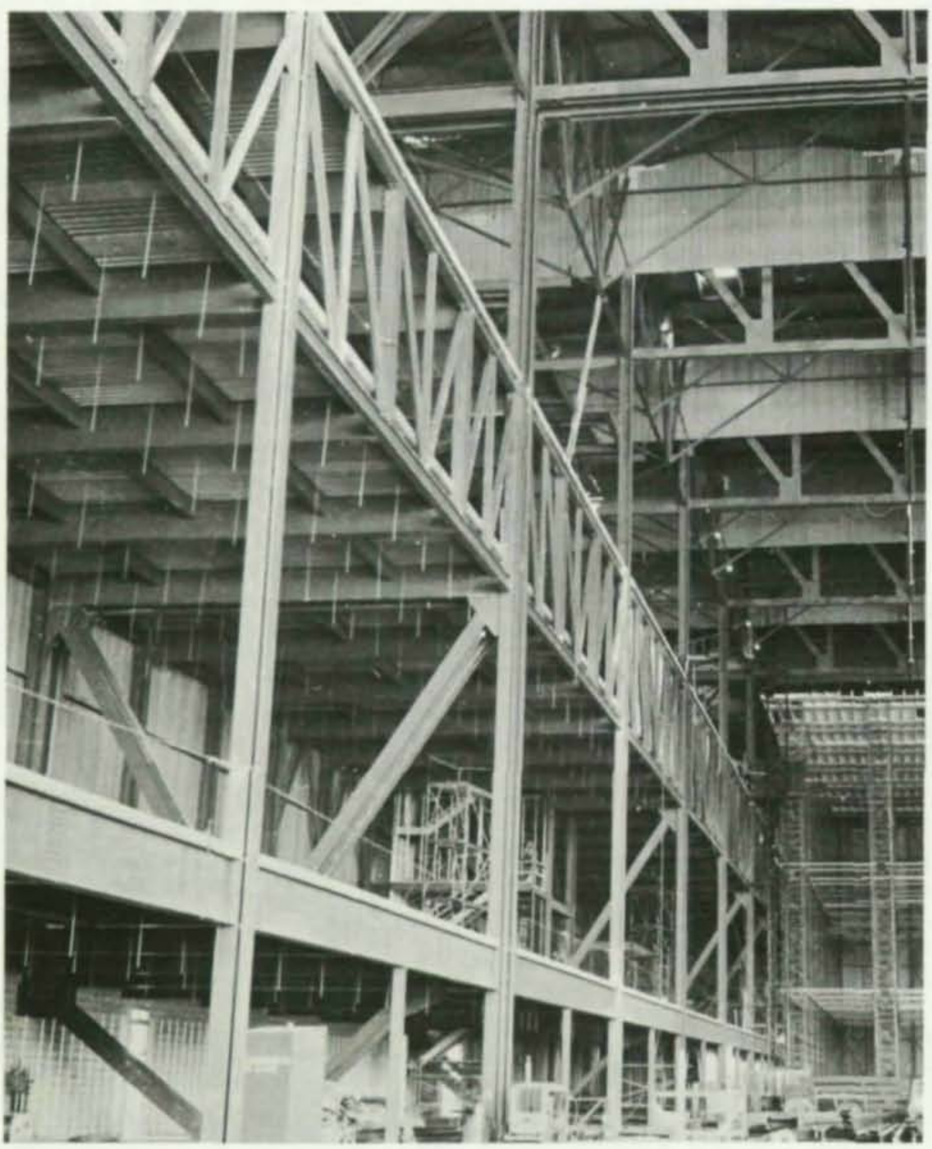
The roof trusses are provided with a horizontal bracing system at the lower chord level. The building is braced laterally by rigid frames and K-bracing at alternate columns of the 900-ft exterior wall and the mezzanine. Specifically, the lateral resisting system is designed for 25 psf wind load, 13% G seismic load, and 100°F temperature change.

Further augmentation consists of three mezzanine levels, 40 ft wide x 900 ft long on the north and south walls of the building, and includes a bracing element utilizing a conventional floor deck and beam system. The east and west walls open to a maximum of 510 ft with fourteen 85-ft high sliding doors.

Steel Fabrication

The roof framing, all A36 steel, weighed 13.2 lbs per sq ft. Structural steel tonnage amounted to 12,500 tons for the structure, hangar doors, and mechanical platform catwalk.

The pockets receiving the huge doors at each end of the structure are accentuated by use of material and color giving a "book end" effect. A space 11 ft high on each side of the building, running its full length between the "book ends," is constructed of concrete block and painted dark brown. This is designed to give the building a base and allow for future doors, windows, piping, etc. to penetrate as needed. Above the base is a clean, simple panel of insulated metal siding approximately 107 ft high x 900 ft long.



Desert Winds and Temperatures

Measures were taken to guard against the desert's 100 mph winds, 100° summer weather, and below-freezing winter temperatures. Specifically, the large (85-ft) sliding doors at the end of the structure deflect up to 8-in. during maximum winds. The doors are spaced far enough apart to operate during a heavy windload and are provided with seals to prevent wind and dust from entering the structure. Rigid wall and roof insulation were used to compensate for hot and cold weather. The building will respond to temperature changes with expansion joints allowing 6-in. of movement due to either expansion or contraction.

Architect:
William L. Pereira & Associates
Los Angeles, Calif.

Structural Engineer:
Brandow & Johnston Associates
Los Angeles, Calif.

General Contractor:
William Simpson Construction Company
Los Angeles, Calif.

Steel Fabricator:
Bethlehem Steel Corporation
Bethlehem, Pa.

LIFTING THE GLASS AT BRANDYWINE

by Arthur Siegel

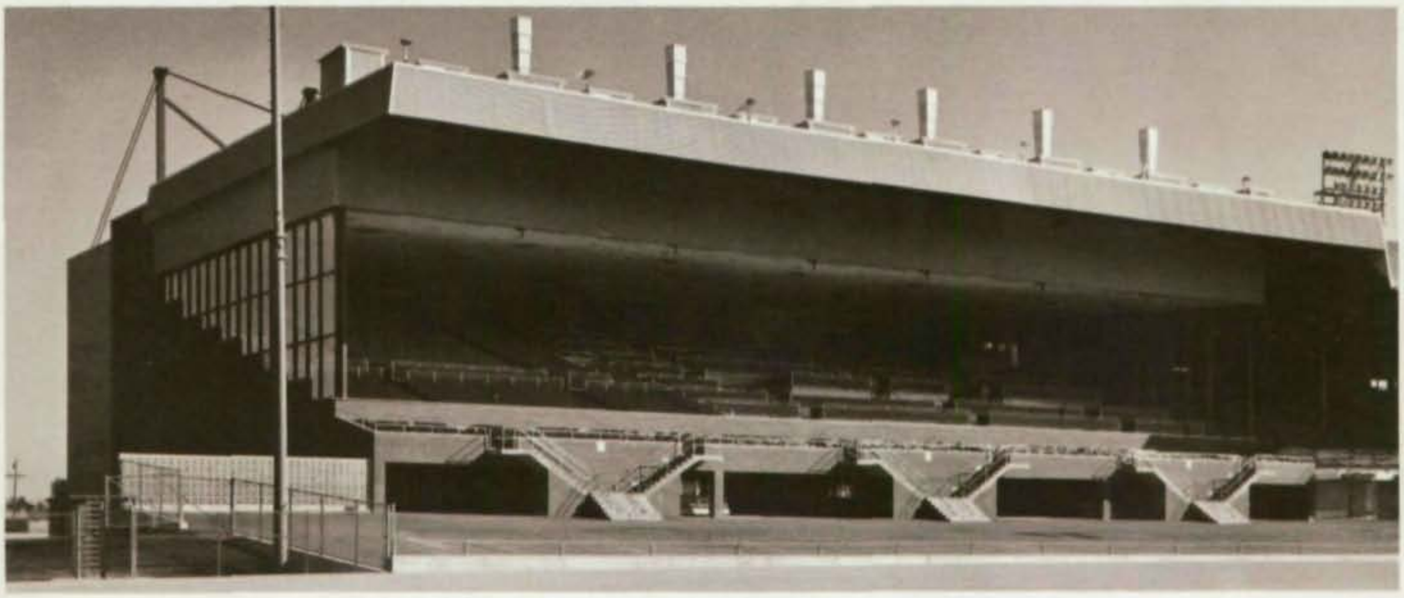
While horses can run on tracks wet and muddy or dry and fast, racing fans prefer winning or losing in creature comfort. The owners of Brandywine Raceway, near Wilmington, Delaware, wanted to furnish maximum comfort in the new clubhouse by providing a heated and air-conditioned enclosure to accommodate some 2,500 spectators. However, they also wanted to be able to move the windows out of the way so that spectators could have a completely unobstructed view of the track whenever weather conditions were suitable. Architect Lionel Levy found the solution to this unusual problem in an ingenious movable window system.

Arthur Siegel, Associate of Robert Rosenwasser, New York, N. Y., was in charge of the structural design of the project.

Full Visibility Required

To achieve the desired open view of the track, each window panel can be raised into the ceiling space above the viewer's sight line. Once the window panels are out of sight, any remaining columns would represent an "eyesore" for this sophisticated window system. Therefore, the architect decided that the front row columns should also be made movable; they are raised into a telescopic housing above the roof. As a result, these columns no longer carry any roof load but serve only as mullions and tracks for the movable windows.

The glazing consisted of 4-in. x 8-in. steel tubes welded to form a rectangular panel 9 ft x 28 ft. Three lights of ½-in. tempered glass, separated by ¾-in. x 8-in. deep steel stiffener plates, were set in the panel. Three such units, suspended at different levels from the roof, offset each other in a horizontal plane. The resulting stack was 27 ft high and 28 ft long. Seven such bays, 28 ft o. c., make up the 196-ft overall length of the clubhouse.



Cable-Supported Roof

Because of the architectural restrictions, it soon became obvious that a cable-supported system would be most economical for the roof structure. Essentially, the cable system replaces the front columns as load bearing supports.

Eight 36WF girders are spaced 28 ft o. c. Each girder is supported on the outboard or track side by a pair of 2⅞-in. diameter cables which pass over a 24-ft high mast and then down to a back row of columns 54 ft away. Standard tar and gravel roofing of 2¾-in. concrete plank is supported by 14B steel filler beams framing into the girders.

"Breathing" Roof

The cable assembly includes a fixed bearing socket on one end and a tensile socket on the other. This tensile socket is fitted with a threaded stud bolt for field adjustments.

The mast assembly consists of a 14WF127 with a grooved saddle on the top and a rocker plate on the bottom. Since the cable changes length or "breathes" during temperature and load variations, the rocker permits axial load on the mast at all times. One of the problems caused by the "breathing" action of the cables was the vertical translation imparted to the window panels. As much as 6-in. vertical movement due to live load is anticipated. This movement is attributable to the small angle which the 145-ft long roof cables form with the roof girders, resulting in very large vertical displacements for correspondingly small changes in cable length. This amplification factor is a 4 to 1 ratio.

Another problem caused by this large amplification factor involved the setting of the outboard roof girder. The elevation of the upper sight line had been based on dead load conditions. Knowing that the vertical movement due to dead load amounted to 10-in., the girders were set with this initial upward displacement. To physically accomplish this, the cable lengths were marked during the prestretching operation. Thus, the actual elongation of the cables due to dead loads was simulated prior to installation.

Similar problems involving the "breathing" roof were present for the non-moving side glazing. Along the sides a brick parapet follows the rake of the stepped seating. Accommodating the moving roof and simplifying the mason's work necessitated making the bottom of the mullions stationary. Vertical slip joints are provided at the mullion tops where they connect to the roof girders.

A clear roof expanse of 107 ft is achieved when the windows are fully open and the telescopic columns are "up." This can result in an appreciable wind pressure on the roof due to combined suction and lift. A heavy stone concrete roof plank was used in order to maintain full tension and safeguard against flutter in the cables.

Steel Erection

To facilitate erection of the roof steel, the main girder was cantilevered toward the track 20 ft beyond the mast line. Steel was erected from the hold-down line to the cantilever end without the use of any cables. The remaining 87 ft of the girder was then spliced using the cable and mast assemblies. This was followed by placement of the roof beams and plank.

Due to the large overhang of the roof and relatively short back distance, the anchor line receives a large upward force which cannot be counterbalanced by the dead load of the roof, floors, and exterior walls. To provide the necessary tie-down force, the foundation wall for the cellar was made 4 ft-4 in. thick.

The foundation for the clubhouse consists of caissons and footings resting on Brandywine blue granite.

Architect:
Lionel K. Levy, Architect
New York, New York

Structural Engineer:
Robert Rosenwasser
New York, New York

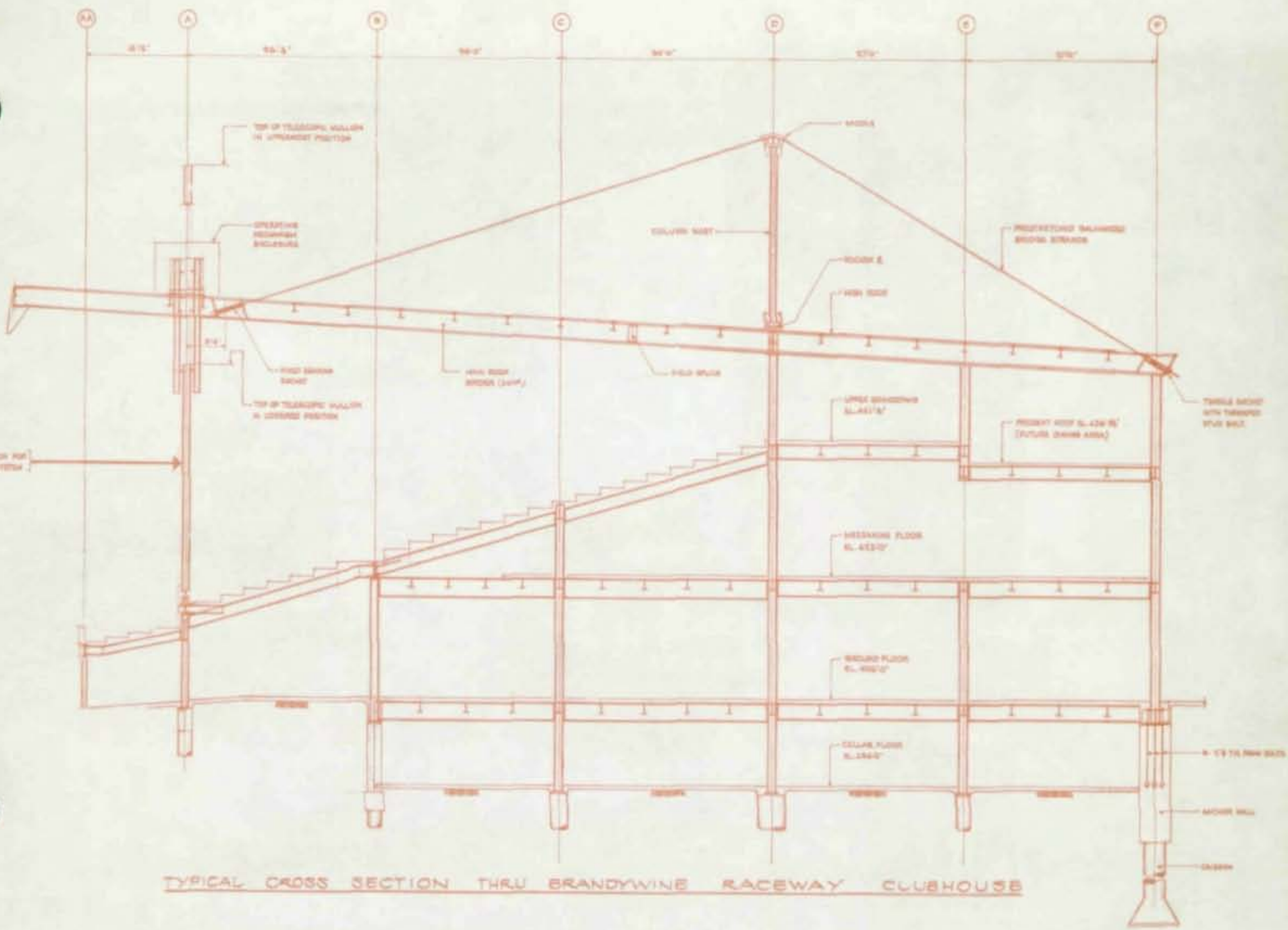
General Contractor:
Ernest DiSabatino & Sons, Inc.
Wilmington, Delaware

Steel Fabricator:
The Belmont Iron Works
Eddystone, Pa.

◄

View of roof shows telescopic mullions in up position and gear boxes which raise and lower the windows and mullions.

Pairs of roof supporting cables rest saddles atop 14WF steel supporting mast.



TYPICAL CROSS SECTION THRU BRANDYWINE RACEWAY CLUBHOUSE

Architect:
Robert E. Alexander, FAIA, and Associates
Los Angeles, Calif.

Structural Engineers:
Parker, Zehnder & Associates
Los Angeles, Calif.

John A. Blume & Associates, Engineers
San Francisco, Calif.

General Contractor:
William Simpson Construction Co.
Los Angeles, Calif.

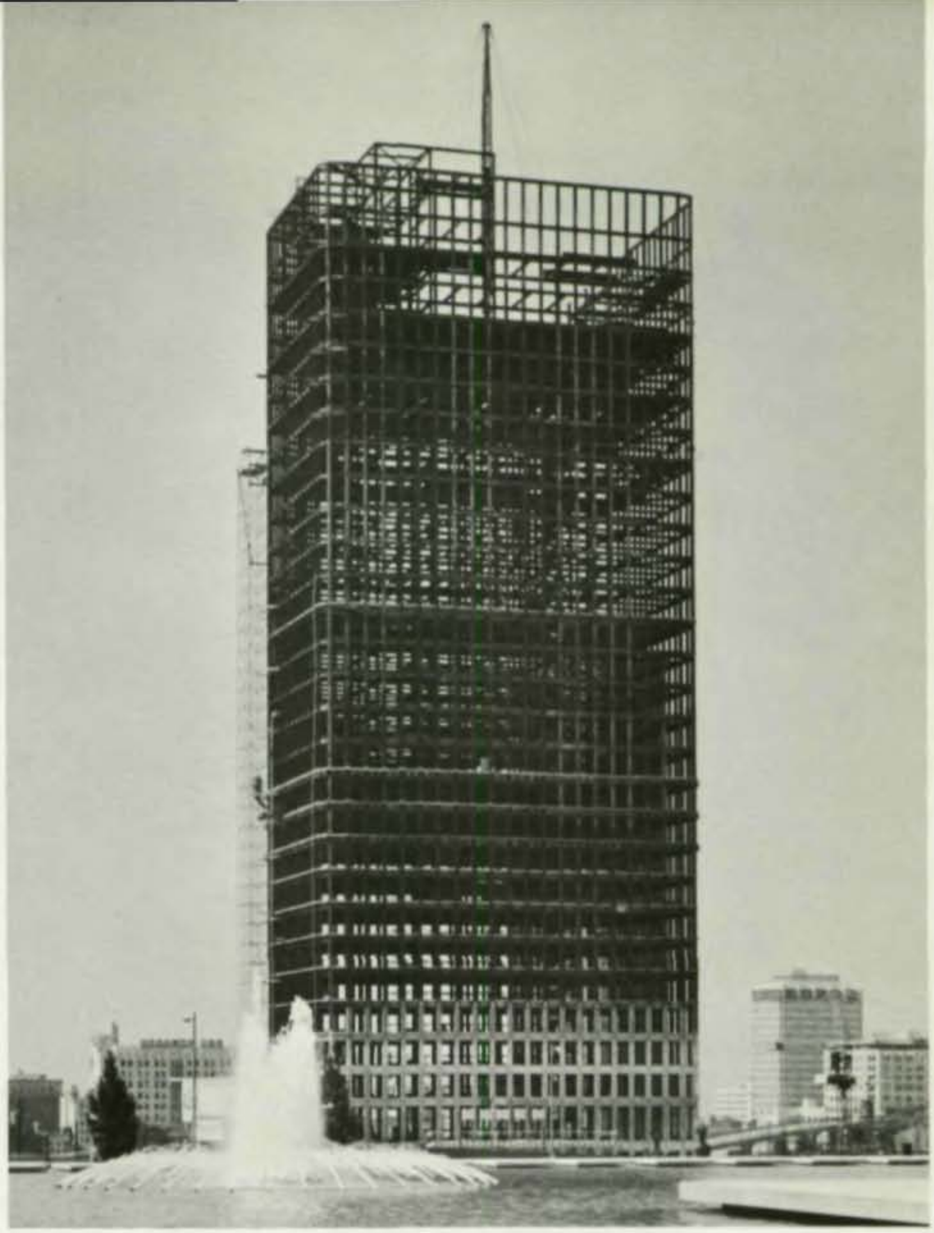
Steel Fabricator:
American Bridge Division—
United States Steel Corp.
Pittsburgh, Pa.

BALANCED DESIGN FOR APARTMENT TOWER

by Robert Alexander and Arthur Parker

The design of the new 32-story steel-frame residential apartment building now going up in the Bunker Hill area of Los Angeles represents the simultaneous achievement of strength, economy, and a strong design that flows naturally from the structural system without adding superficial architectural treatment.

This structure is one of three units that will comprise the first phase of the Bunker Hill Towers, a \$60-million residential complex being built on a site of 13.7 acres. The other two buildings will be 19 stories high.



Structural Systems Studied

Two important factors had to be considered in selecting the structural system for this 32-story building: budget limitations and the need for providing seismic as well as wind resistance for a high rise apartment dwelling.

Detailed analyses of structural systems used in other tall buildings, chiefly that of New York's twin 102-story World Trade Center structures, were made.

Finally, three alternative proposals for a structural system were fully explored:

1. Conventional moment resistant frames with all exterior and interior columns participating in frame action.

2. Moment resistant frames in outside wall lines only, with exterior columns spaced at 10 ft o. c.

3. Moment resistant frames in outside wall lines only, with exterior columns spaced at 5 ft o. c.

Plan 3 was decided upon with one modification—changing the exterior column spacing from 5 ft to 5 ft-9 in. o. c.

Adoption of the third alternative resulted in a number of savings in non-structural elements:

Freedom for more efficient architectural planning—The small exterior module readily accommodates reasonable room sizes in widths of two, three or four bays. In addition, the location of interior columns need have no relation to modular exterior spacing, since there are no through building rigid frames.

Uniform story height and reduced total building height—The total height of this 32-story building would have been increased by more than 20 ft had Plan 1 been adopted. The reduction in story height is made possible since all deep rigid frame girders occur in exterior walls where they are enclosed by window heads below the ceiling. All interior framing then is of uniform minimum depth, supporting vertical loads only.

Reduced cost for enclosure of building—After the exterior columns were fireproofed and given a weatherproof covering, the outside walls were completed by placing 4-ft wide window sections.

Ultimate Resistance to Lateral Forces

Several structural characteristics of the unusual design concept increased the building's ultimate resistance to lateral forces at an economical cost. Among them are: perfect symmetry of the structure; the highly redundant lateral force system, (there are 72 exterior columns each carrying relatively small vertical loads); the great torsional strength which results from locating all rigid frames in exterior wall lines; the increased member and joint reliability—the heaviest column or girder flange in the rigid frames is only 1⅛-in. thick, thus reducing the probability of member defects, such as laminations and inclusions, and greatly simplifying the attainment of uniformly reliable welded joints. Additionally, the close spacing of the outside columns and girders permitted the shop fabrication and field erection of entire sections four floors high and three columns wide, a time-saving departure from conventional construction.

As to safety, this design has been checked both statically and dynamically for as thorough an analysis as any structure known.

The weight of the structural steel is 2,982 tons, and averages 16.8 psf at a cost of \$3.83/sq ft, comparing favorably with an estimated 20.3 psf and up to \$4.50/sq ft for conventional design.

Robert Alexander is a principal in the firm of Robert E. Alexander, FAIA, and Associates, Los Angeles, California.

Arthur Parker is a principal in the firm of Parker, Zehnder & Associates, Los Angeles, California.

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