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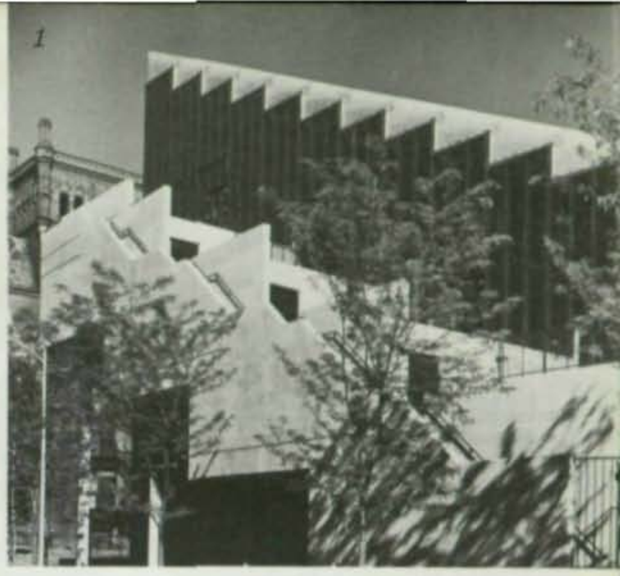
MODERN STEEL CONSTRUCTION



MODERN STEEL CONSTRUCTION

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CONTENTS

1973 PRIZE BRIDGE COMPETITION

Entries are invited for the 45th Annual Prize Bridge Competition to select the most beautiful steel bridges opened to traffic during the calendar year 1972.

The members of the 1973 Prize Bridge Jury are:

Ralph R. Bartelsmeyer *Acting Federal Highway Administrator, U.S. Department of Transportation, Washington, D.C.*

Arthur L. Elliott, F.ASCE *Bridge Engineer—Planning, Division of Highways, State of California, Sacramento, California*

Egbert R. Hardesty, F.ASCE *Hardesty & Hanover, Consulting Engineers, New York, New York*

Thomas E. Stelson, F.ASCE *Dean, College of Engineering, Georgia Institute of Technology, Atlanta, Georgia*

Charles W. Yoder, F.ASCE *President-elect, American Society of Civil Engineers; C. W. Yoder & Associates, Milwaukee, Wisconsin*

Entries must be postmarked prior to May 26, 1973 and addressed to the Awards Committee, American Institute of Steel Construction, 101 Park Avenue, New York, New York, 10017.

1973 NATIONAL ENGINEERING CONFERENCE

The 25th Annual AISC National Engineering Conference will be held on May 10 and 11, 1973 at the Bellevue Stratford Hotel, Philadelphia, Pennsylvania.

Leading authorities in steel design, research, and construction will meet to exchange ideas and information about the latest developments in these fields. This conference is a "must" for anyone who designs structures.

1973 FELLOWSHIP AWARDS

Four engineering students have been awarded $3,000 fellowships in the 11th Annual Fellowship Awards Program. The program is designed to encourage expertise in the creative use of fabricated structural steel.

Robert D. Anderson *University of Colorado*

Michael F. Garrett *University of Minnesota*

James C. Hays *University of Pittsburgh*

Maria M. Nagy *Massachusetts Institute of Technology*

STEEL SKYLIGHT SPANS STEPPED STUDIO

Architect:
John Andrews, Architects
Toronto, Canada

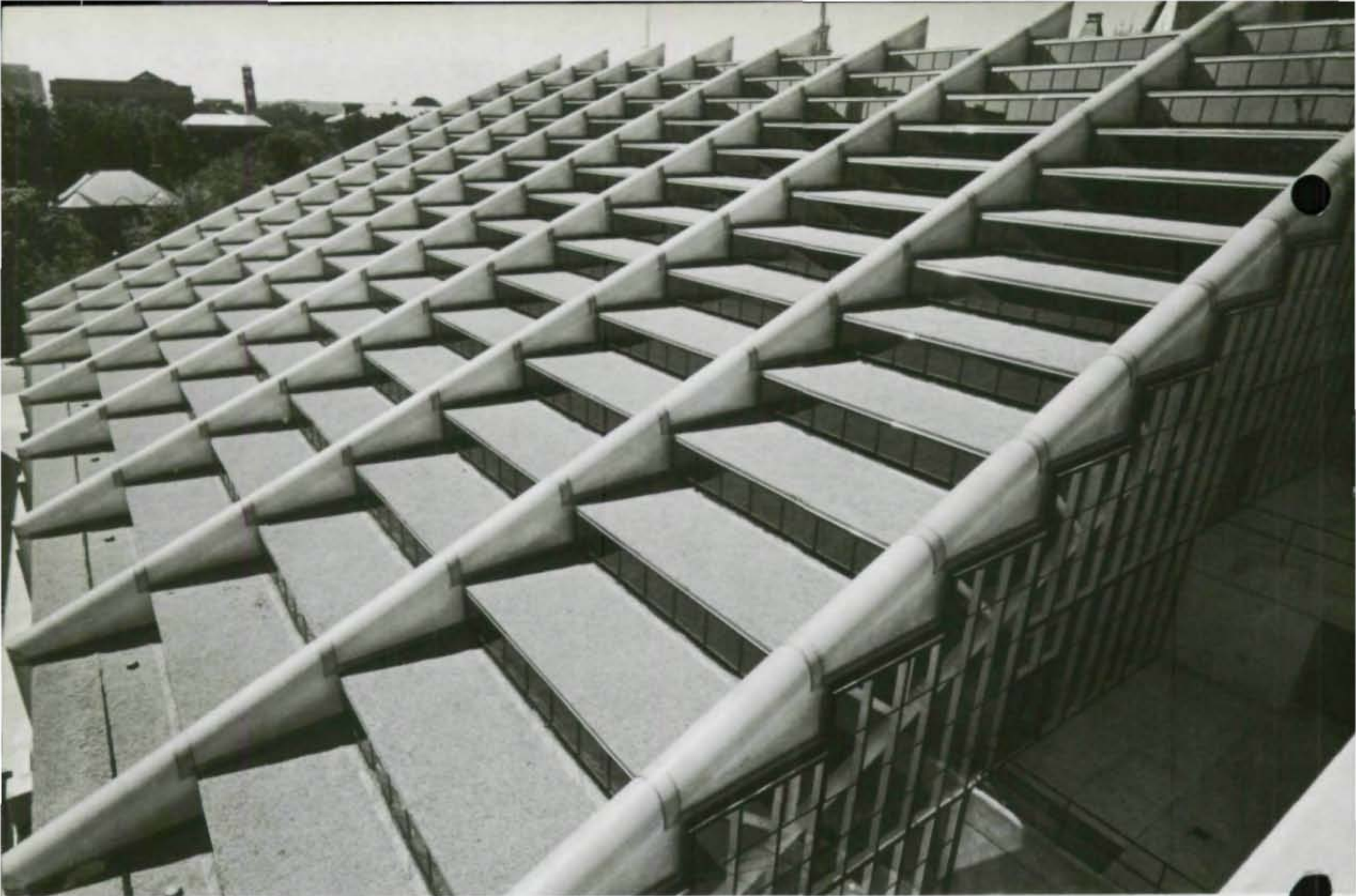
Structural Engineer:
LeMessurier Associates
Cambridge, Massachusetts

General Contractor:
J. Slotnik Co.
Boston, Massachusetts

Steel Fabricator:
West End Iron Works
Cambridge, Massachusetts

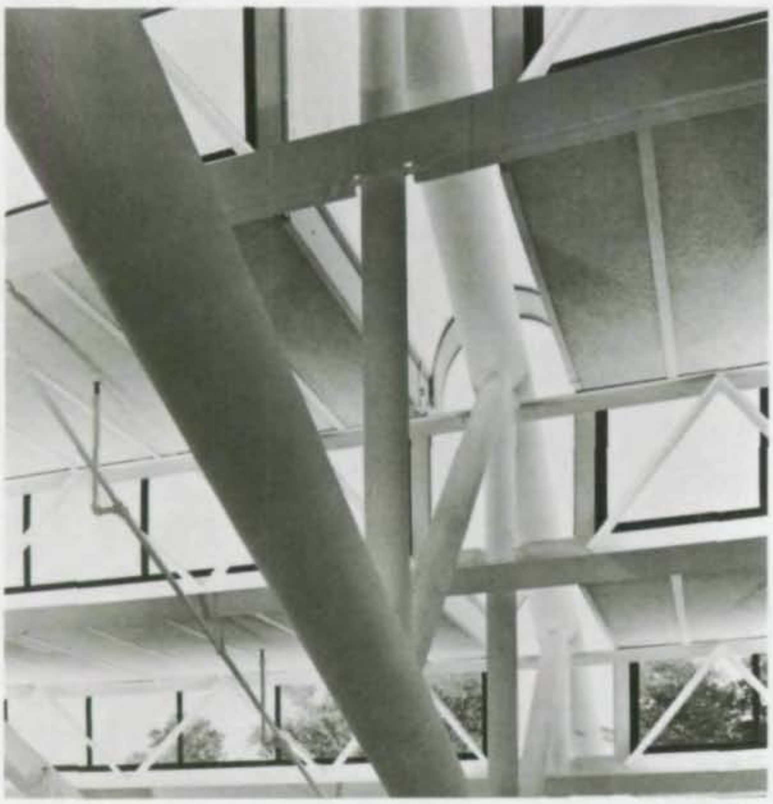
Gund Hall, the new and unusual building that houses all programs of Harvard's Graduate School of Design under a single roof, reflects a clear architectural concept. Basic to the building's plan is the idea that students in all of the school's design disciplines—architecture, city and regional planning, environmental studies, urban design, computer graphics, spacial analysis, landscape architecture—should be exposed to each other, mingle freely, and work together. The key space in the structure is a single large studio area designed to promote such a student mix and to serve as the focal point of all support spaces.

The building is a 4-tier, stepped structure, 244 ft long by 158 ft wide by 80 ft high, providing 160,000 sq ft of interior space. The studio is a single integrated space on four stepped levels, and resembles a simple loft space under a huge inclined steel and glass saw-tooth skylight. The stepped design provides 50,000 sq ft of studio space in a single clear span area, despite the limited size of the building site. At each level, office and seminar areas are located immediately adjacent to or under the studio area, wrapping completely around the building. A 400-seat auditorium, a library, and a technology workshop are located at ground level.



Above: The nine A36 steel pipe trusses supporting the tiered roof were erected in one week.

Below: Interior view of truss.



Steel Pipe Trusses

The cascading studio area is spanned by nine inclined A36 steel pipe roof trusses, spaced 24 ft o. c. Each truss is 11 ft deep by 134 ft long and weighs 13 tons. Top and bottom chords are nominal 12-in. dia. pipes with wall thickness ranging from 0.344 in. to 0.625 in. Web members are 6-in. dia. pipe sections with wall thickness from 0.156 to 0.344 in.

To stabilize the trusses and to resist lateral loads, the roof's two end bays are X-braced with ⅞-in. and 1¼-in. wire ropes. To accommodate movement, the lower end of each main truss rests on a Teflon-coated bearing plate.

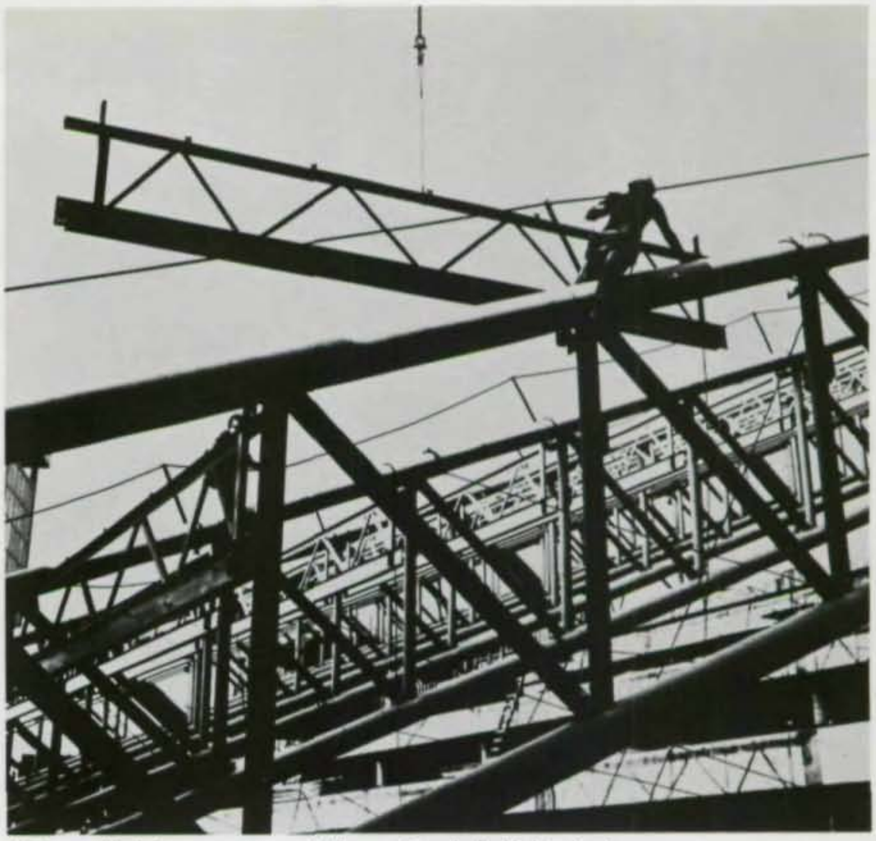
Each interior web vertical of the nine primary steel pipe trusses supports 24-ft long by 3 ft-6 in. deep intermediate trusses, fabricated of A36 steel channels, angles, and tees. In turn, these smaller trusses and rows of steel bulb tees support the 15-step roof and the skylight glazing.

The exposed steel primary trusses are coated with a ⅛-in. thick layer of white intumescent mastic that not only provides a ¾-hour fire-resistance rating, but also gives a pleasing finished appearance from below. This paint, when exposed to dangerously high temperatures, forms a thick layer of incombustible insulation that effectively protects the steel from fire damage.

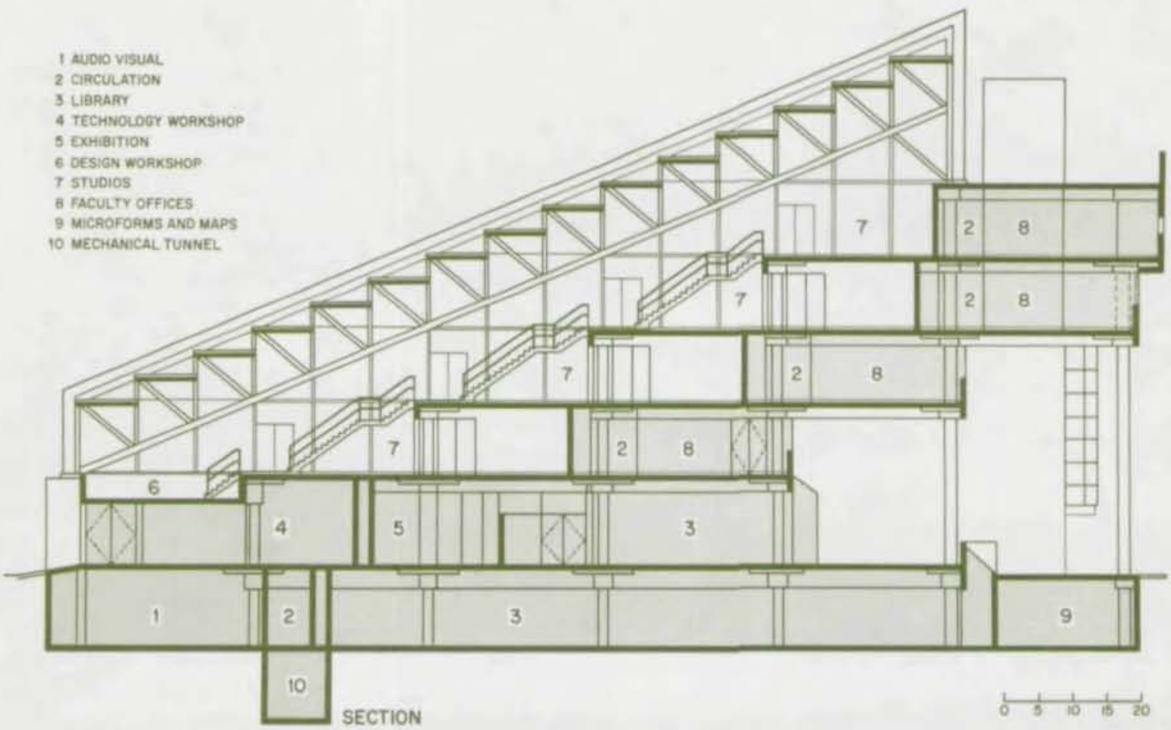
Skylight Glazing and Roofing

A variety of performance reflective glass — supplied on an experimental basis by the glass manufacturer — yields a relatively high shading coefficient in the sun to keep the studio from being too hot. Clear glass is used along with the reflective glass, depending on the exposure. In effect, the glass forms the "riser" of each of the 15 steps of the sloping roof, while a 4-in. thick bonded fibrous roof panel forms the horizontal "tread."

Special prefabricated fiberglass covers, curved to fit over the tubular steel top chords of the nine main trusses, protect the framing from the weather while their transluscence adds to the skylight effect in the studio below.



Intermediate trusses support the roof and skylight glazing.



Cross section of Gund Hall, Harvard University's Graduate School of Design at Cambridge, Mass.

RAISING THE ROOF AT BUFFALO

by Harold P. Roberts

The Buffalo Memorial Auditorium, orginally constructed in 1939, no longer provided adequate seating to meet the needs of present day major league sports and other entertainments. As part of an overall modernization program, it was decided to expand the seating capacity from 10,353 to 15,172 by building a second balcony containing 4,819 additional seats.

After considerable investigation by our architectural firm, and our structural consultants, it was elected to literally "raise the roof" — to raise the entire roof as a single unit from existing steel. A total lift of 24 ft was required for the installation of a second balcony to support the additional seating.

According to available engineering records, this lift is the largest in area thus far recorded for the single respositioning of an architectural element.

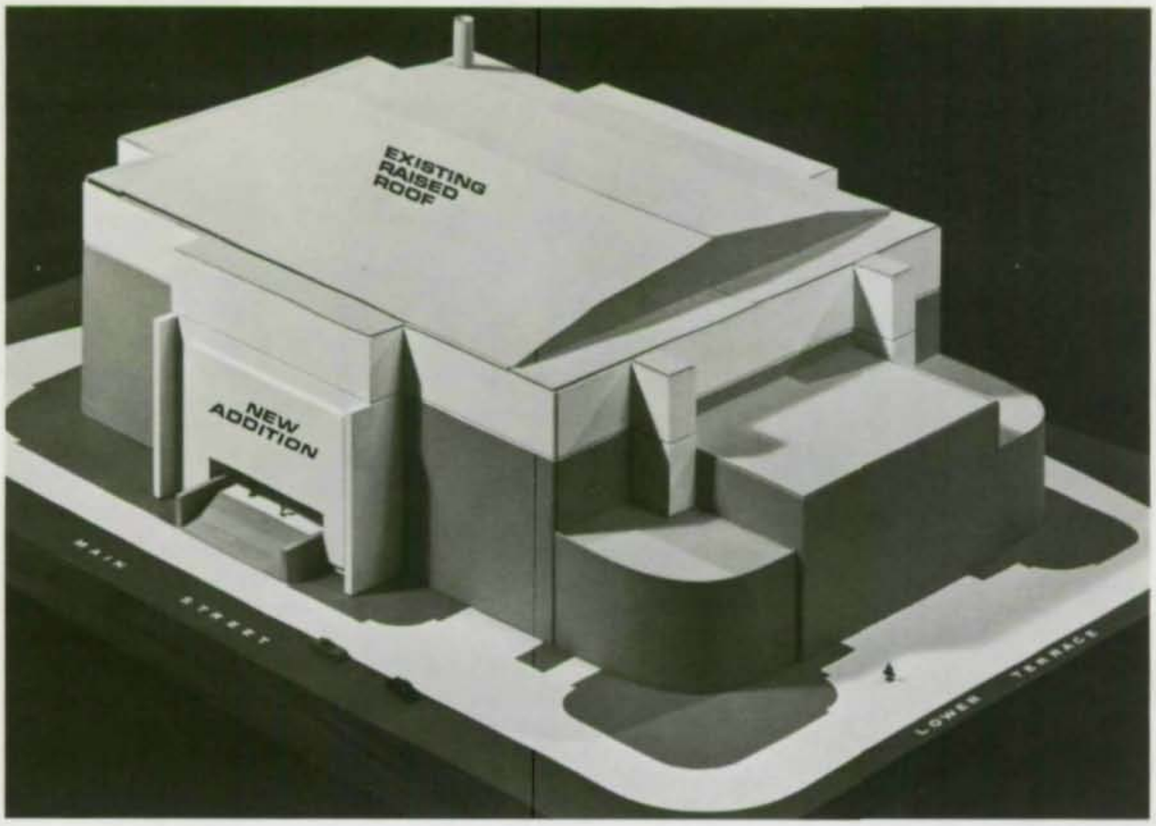
Mr. Roberts is a partner in the firm of Pfohl · Roberts · Biggie, architects and engineers, Buffalo, N.Y.

Architect:
Pfohl · Roberts · Biggie
Buffalo, New York

Structural Engineer:
McKaig Rupley Bahler
Buffalo, New York

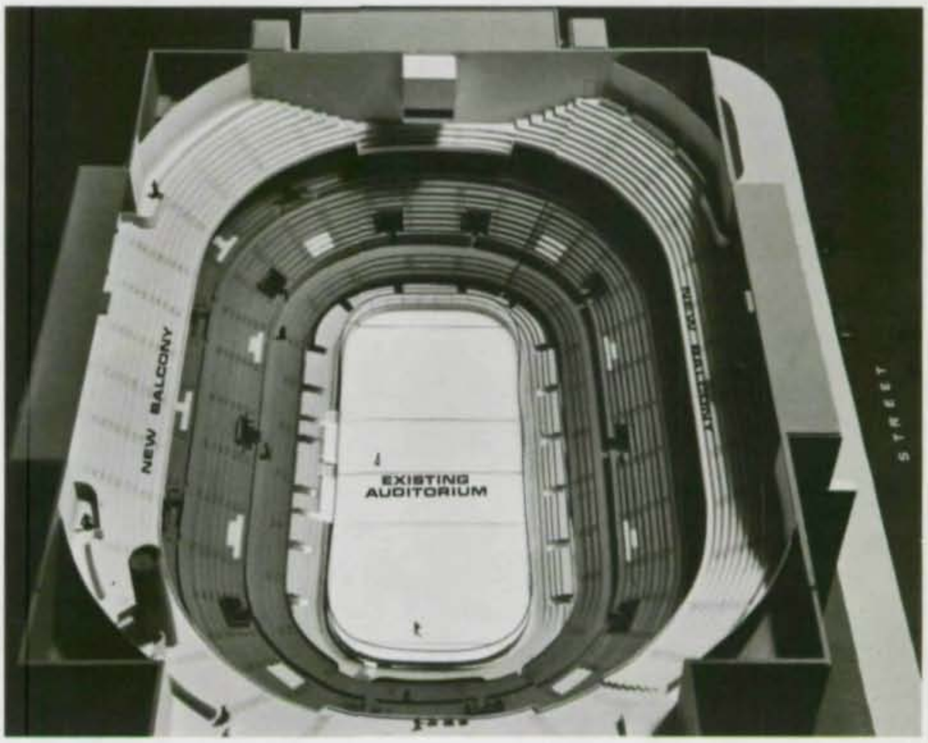
General Contractor:
Siegfried Construction Company, Inc.
Buffalo, New York

Steel Fabricator:
Levinson Steel Company
Pittsburgh, Pennsylvania



The roof structure, measuring 260 ft across by 276 ft long, consisted of 11 bays with trusses arranged on 25 ft-2-in. centers. The entire slab involved 71,600 sq ft and was equivalent to 1.645 acres—about the size of a football field. It was estimated that the roof slab weighed approximately 2,200 tons.

The actual lifting of the roof was a climax to many weeks of careful preparation. Existing building steel was extended upward so that the new building columns could be used as a support for the lifting jacks. Holes were cut in the roof to facilitate the steel extension. The original design was ideally suited for repositioning the roof, since each end of each building truss was supported by a single 4½-in. steel pin extending through both the truss and the square building column. To expedite the removal of these pins at the actual time of the lift, each pin was removed and replaced with a slightly undersize pin for manual removal later on. Although it was anticipated that some difficulty might be involved in removing the original support pins, liberal lubrication and the use of hydraulic jacks, enabled the pins to be removed without difficulty. During the pin change, the roof itself was supported on the lifting jacks which had been positioned earlier.



Above: Model of completed project.
Below: Interior of model.

Existing steel is extended upward, then roof is raised to new position.

Extended steel columns support the lifting jacks.

As part of the preparation, the existing building steel was given additional support through a complex of guy cables. This, in conjunction with other temporary steel, provided the additional rigidity of the supporting skeleton, to not only facilitate the lift, but also to insure an adequate safety factor in the event of misalignment during the lift or adverse weather.

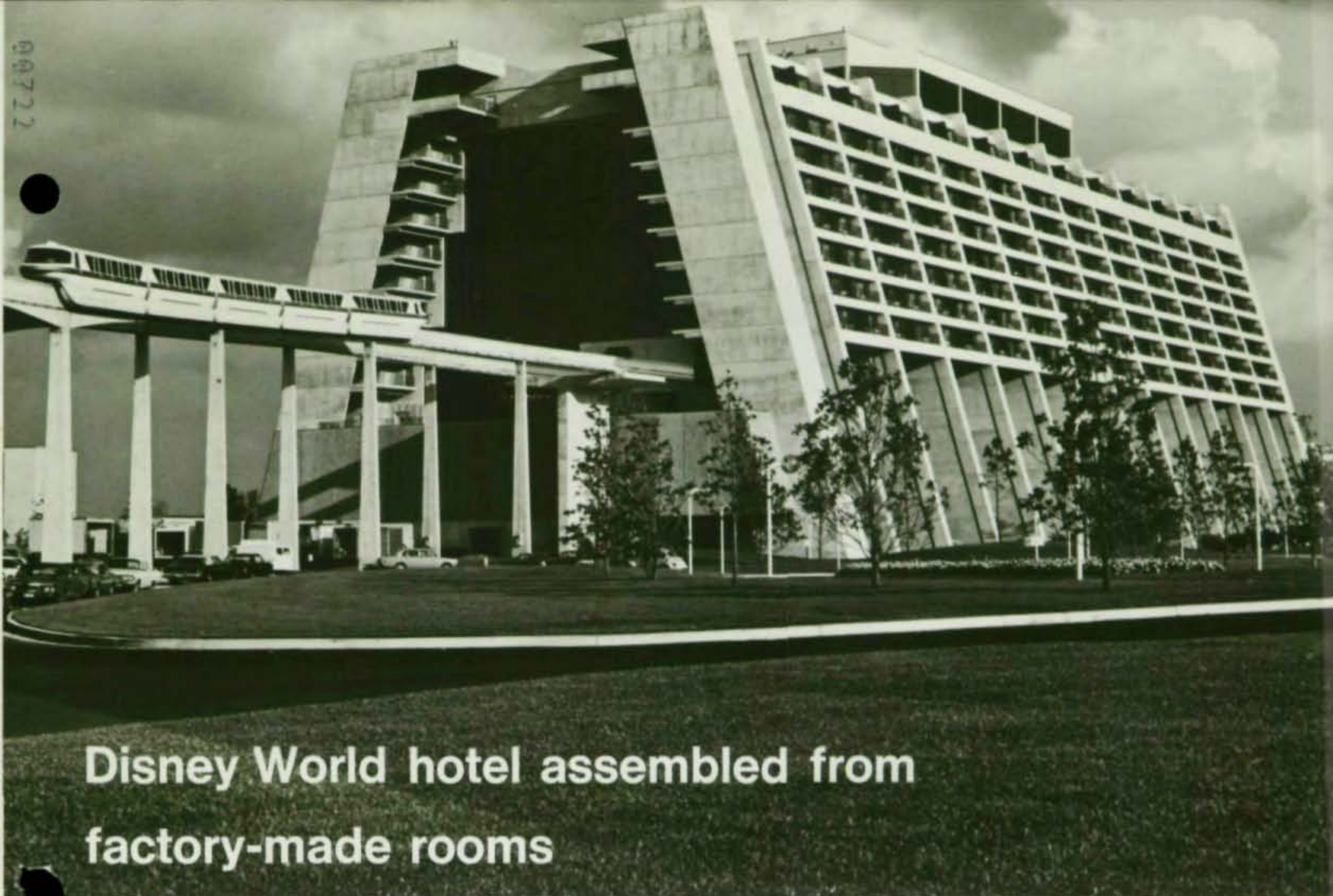
The hydraulically-operated lift jacks were supported above the roof on top of the new building steel. A pair of jacks, each equipped with two lifting screws, was assembled on each column. Each of the individual assemblies was connected to two hydraulic consoles. Two separate assemblies were used for each side of the roof and were powered by two diesel engines.

Electronic strain gages were connected to each of the lifting screws to insure against overstressing. Other controls monitored the lifting action of each jack so that the lifting rate at each position would be identical.

The jacks were started up slightly after 8:00 A.M., and each of the column positions were given a short burst to prove out the controls and hydraulic systems. After a few minor hydraulic problems were corrected, and the initial lift of four to six inches was made, the equipment was again checked. (When each of the jacks were actually supporting the roof loading, the temporary undersize pins were removed.) Shortly after 10:00 A.M., the pumps were put into high gear and 95 percent of the lift was made without difficulty.

The roof was within six inches of its final position at 4:00 P.M. At this point, the lifting rate was reduced and the roof was then inched into final position. As soon as the roof truss pin holes lined up with the new position, undersize pins were inserted. The final leveling to accommodate the full-size pins required individual truss positioning.

Following the lift, other supporting steel was immediately erected to insure rigidity of the extended structure, while at the same time enabling workmen to proceed with other phases of the building expansion.



Disney World hotel assembled from factory-made rooms

Four years ago WED Enterprises, the engineering and master planning division of Walt Disney Productions, was seeking a new and unconventional method for constructing five luxury hotels at the new Walt Disney World complex. WED and architect Welton Becket & Associates developed specifications for a **modular** hotel room. Selected industrial firms were invited to develop a modular hotel room construction system.

U.S. Steel responded by forming a task force with people from USS Realty Development, American Bridge Division, U.S. Steel Homes Division, Construction Marketing and Applied Research Laboratory. The USS task force established several objectives. Among them:

- Weight of the room unit had to be kept to a practical minimum for ease of handling and transportation.
- Construction materials had to facilitate the assembly line process and make for easy handling by shop craftsmen.
- Modules had to be economical.

U.S. Steel developed a room unit design of a three dimensional steel frame using a maximum of "dry" materials in the finishing process on the shop assembly line. A prototype model was developed to analyze architectural and structural detail and test for structural integrity, handling and transporting, sound transmission, and vibration. Underwriters, Laboratories helped test for fire endurance and performance under smoke conditions. Finally, USS and Disney decided to use this unitized hotel room technique to construct the Contemporary and Polynesian Village Theme Hotels. The Realty Development Division of U.S. Steel was selected general contractor.

Building Blocks

The design of the high-rise Contemporary Hotel was guided by three factors:

- The flat and "open" topography of the Central Florida region suggested the hotel should create for its guests a vast and "open" environment, eliminating any feeling of confinement, while providing protection from the Florida heat, humidity and rain.
- Disney's desire to create an exposition-recreation and festive atmosphere, making the hotel an attraction in itself.
- The desire to include the Disney monorail as part of the Contemporary Hotel, thus enhancing the hotel's festive environment while providing a transportation link to the other hotels and the Disney Theme Park.

The result was a 14-story A-frame structure with rooms terraced up both sides of the A. This formed a vast nine-story-high interior space open to the outdoors through glass endwalls and a skylighted roof. The monorail passes through the interior of the hotel.

The structure is 184 ft high, 220 ft wide at the base, 468 ft long. It contains 1.4 million sq ft of space. Nine levels of guest rooms topped by a penthouse skyroom restaurant rise from a massive four-level base containing the hotel's major convention and public

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spaces. An additional 621 guest rooms in the form of several connected three-story buildings radiate from the central high rise element. The hotel had to be completed by October 1, 1971 — only 18 months were allotted to prepare working drawings and specifications and to construct the facility.

Foundation

The high-rise building is supported entirely on piles. Annex units and other miscellaneous structures are supported on conventional spread footings. The pile is a driven 14-in.-dia. steel pipe filled with concrete. The piles were driven into the limestone soil from 40 ft to over 200 ft. The piles are designed for 10 tons lateral load and 75 tons axial load. Reinforcing in the top part of the pile resists lateral wind loads.

After installation of piles, pilecaps, and struts, two slipformed concrete elevator cores were built to full height. When the core had reached its full height, erection of the 7,000 tons of structural steel began. At this stage, construction accelerated — only 11 months from start of steel erection to opening date in October 1971.

Erecting the A-frames

To frame the large atrium area and support the hotel's basic building blocks, 13 pairs of structural steel A-frames, spaced 33 ft apart, slant upwards from the ground to a flat roof. Each steel A-frame is 220 ft at the base, 135 ft at the top and 150 ft high with vertical bents 26 ft wide and the top truss 15 ft deep. Each was constructed of fabricated plate box and wide flange sections with the chords made of 18-in. by 26-in. tubes.

Assembled in the field and erected in five pieces, the steel leg sections were placed one at a time with 200-ton crawler cranes and with the aid of a temporary falsework bracing system starting at the center of the hotel at the concrete core. With the two sloping leg sections in place, the preassembled roof truss sections were hoisted into position. The temporary bracing was then moved to the adjacent position and the sequence was repeated, working from the center core toward each end of the building until all 13 primary frames were in place. By the time the two ends of the structure were reached, the connecting steel framing for the first four levels had been completely erected, and the steel floor deck and concrete slab construction was already in progress.

All structural steel was A36. Field connections were made with high-strength bolts (ASTM A490 and A325). Structural steel was fabricated by U.S. Steel's American Bridge Division at plants in Ambridge, Pennsylvania and Orange, Texas.

The 13 A-frames were computer analyzed for all different loading combinations of wind, temperature variation, and vertical, dead, and live loads. The temperature variation is based on an 80°F differential (±40 degrees). The wind load on the lower part of the building is transferred directly to the floor diaphragms and from there to shear walls; the A-frames are therefore freed at the three floor levels. A post-tensioned concrete strut under the lower level connects the A-frame bases. At the guest room levels the wind load on the exterior walls is transferred to the A-frames, which are connected at the suite floor level diaphragm. The hotel was designed to withstand wind velocities up to 100 miles per hour.

Thirteen pairs of 150-ft high steel A-frames were erected around a central elevator core.

Preassembled two-room units "plug into" rectangular slots in steel framing.

Placing the Room Modules

Except for some luxury suites on the 13th level, unitized steel construction is used for each hotel room from the 5th floor to the 12th floor on one side of the structure and from the 6th to the 12th floor on the other side. Each room module weighs 10½ tons — one-fifth the weight of equivalent sized modules made of concrete.

The units were equipped on the assembly line, developed and operated by the American Bridge Div., with all architectural interior elements and utilities, including bathroom, lighting, wall coverings, and climate-control fixtures. Next, they were trucked to the site. The modules, were hoisted into place and "plugged into" the building's structural and utilities systems. Erection of the rooms was from the center towards either end of the building and were placed at a rate of seven to 10 units per day. One side of the module is supported by the A-frame and the other side by steel cables pretensioned between the top and bottom steel frame system. The high-rise modules measure 40 ft long, 15 ft wide, and 9 ft high. The guest room vertical loading is resolved into components directed along the inclination of the cables and into horizontal components on the plane of the guest room diaphragms.

The cables are 1¼-in.-dia., 7 by 7 high-strength wire rope having a yield strength of 153 kips, a breaking strength of 187 kips, and a modulus of elasticity in excess of 27,000 kips/in.^{2} Open socket fittings are swaged to the cable ends. Cable pretensioning was obtained by jacking against shoes fitted to the underside of the lower cable girders.

Initial cable pretensions were relaxed at the lower fittings as the cables were progressively loaded by erection of the modules. A principal design effort was directed to determine values of initial pretensioning so that only nominal residual tensions remain at the lower fittings after full dead and service loads were applied.

Endwalls, Stairs, Monorail

During erection of the rooms, construction started on the tubular steel space frames, which support the glazed end walls and have maximum width of 100 ft and a height of 90 ft. The exterior side is enclosed with glass connected to painted steel pipes. Air is provided through horizontal pipe members to accommodate condensation of water on the glass.

The exterior poured-in-place concrete stair-shafts for emergency exits from the suites and guestrooms were built on a slope following the angle of the A-frames. The shafts are free from the A-frames and are connected to the building only at the base and at the suite level diaphragm, a U-shaped channel spanning structurally from top to bottom. The stair flights and landings within the shafts are constructed of reinforced concrete. The steel A-frames were clad with a light buff-colored precast concrete. Spandrels and room exteriors were faced with a matching color porcelain enameled panel.

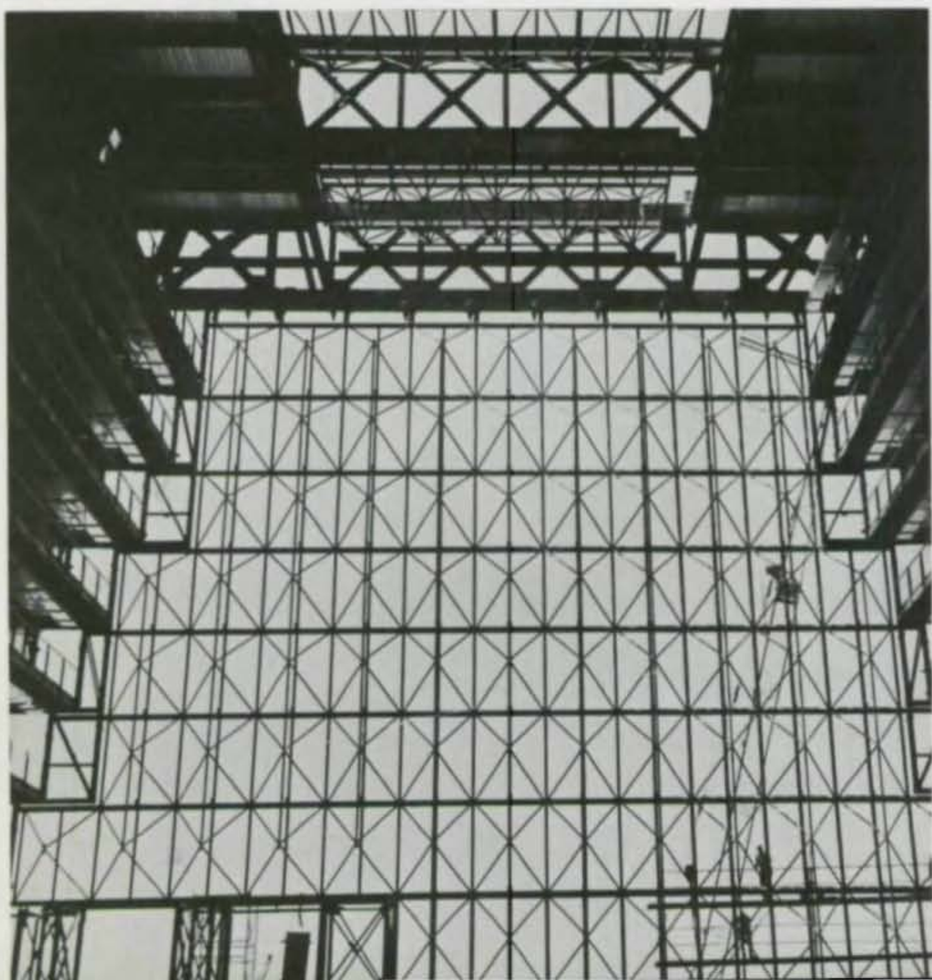
Extending into the large lobby atrium is the monorail, its tracks and platform located at the fifth level. It is made of poured-in-place reinforced concrete supported at 66 ft intervals. The only connection between the monorail and the building is a strut at each support, which links the platform to the A-frame. The strut reduces horizontal deflection of the monorail support. This structural design avoids vibration, since the monorail columns penetrate but do not connect with the lower floors below. The supports rest on a pilecap isolated from the building's column pilecap.

Elevator Core

In the dual central concrete core, five high-speed passenger and three service elevators serve the tower floors. The core consists of two 180-ft high shafts — one 30 by 36 ft and the other 17 by 36 ft. Steel bridges link the core with the room corridors on the guestroom levels. Ends of the bridges are suspended by hangers connected to four concrete girders which cantilever off the core at the skyroom level.

Within the atrium are some of the hotel's dining and lounge areas, shops and spectacular tile mural facing the elevator core. Convention facilities include a ballroom that accommodates 1,200 guests and a large exhibitorium at the lower levels. Crowning the structure on the 14th floor is a spacious octagonal, glass walled skyroom providing a view of Walt Disney World for more than 300 dinner guests.

Tubular steel space frames support the glazed end walls of the hotel.



FIRE IN AN EXPOSED STEEL PARKING STRUCTURE

by Richard G. Gewain

Automobile burning 24 minutes into the test.

Compared to fires in office or apartment buildings, the fires that occur in open-air parking structures pose only a minor threat to life and property. The low fire hazard present in modern parking decks, increasingly recognized by building code authorities in recent years, was conclusively shown by the results of a recent study undertaken by American Iron and Steel Institute.

The AISI study was conducted in three phases: (1) an examination of existing fire test data, (2) an in-depth survey of actual fire experience in all types of automobile parking structures, and (3) a full-scale burnout in a recently constructed exposed steel parking structure.

The low fire hazard found in all three phases of the study can be attributed to several key factors: First, the automobile is the primary source of fuel for a fire in a parking structure, and its combustibles are largely contained within the car itself. Second, there is little chance of an automobile fire spreading even when other automobiles are parked in adjacent stalls; the fire remains in one car, does not develop into a conflagration and is not of sufficient size or duration to bring about permanent deformation or failure of structural steel framing in the fire area. Third, the code-required open wall areas permit rapid dissipation of smoke and heat, providing easy access for fire-fighting personnel.

Mr. Gewain is Chief Fire Protection Engineer, American Iron and Steel Institute, New York, New York.

Existing Fire Tests in Parking Structures

The results of test fires conducted in automobile parking structures — either in actual structures or in mock-ups built solely for the purpose of the test — were studied to determine the level of temperature attained by the steel structural members located within the test area. In three tests conducted in England, a single car was ignited and allowed to burn in an arrangement of nine automobiles. The temperature of the exposed structural steel framing over and adjacent to the fire generally did not exceed 440°F. (In one test, the steel attained a maximum temperature of 680°F.) The tests indicated that an automobile, if left to burn uncontrolled, will consume all combustibles (including the interior upholstery, tires, and fuel in the gas tank) in approximately 45 minutes. The test further demonstrated that the automobile acted as a container, and that there was no communication of fire to adjacent cars, either by flames or a gasoline spill fire.

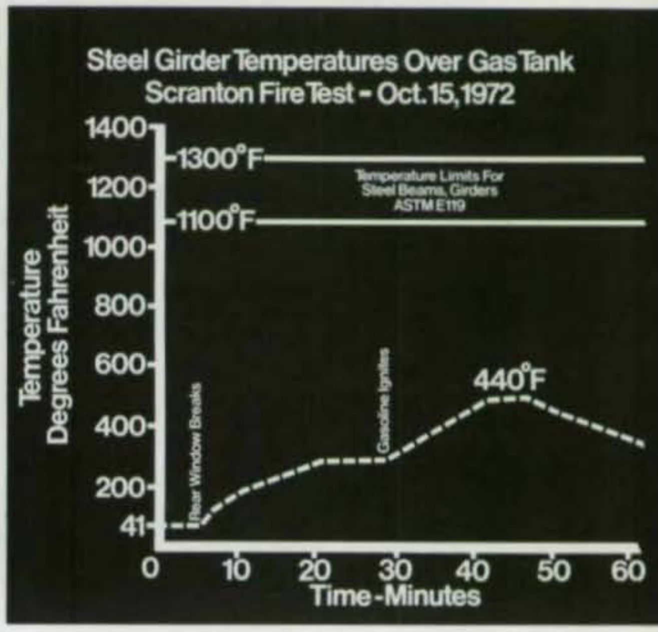
Five fire tests, conducted in Japan, confirmed the results of the British tests, demonstrating that neither the temperature of the exposed steel framing nor communication of fire from the burned car to adjacent ones was a factor of concern. In all the fire tests, by both the Japanese and the British, it was found that an automobile fire is quite sporadic in its flame exposure to both the steel structural members and adjacent cars. The flames continually move about, remaining in one spot for only a matter of minutes. Because of this, it was difficult to locate a sufficient number of thermocouples so that a true flame pattern could be determined by recording air temperatures over the burned car. In the British tests, air temperatures did reach about 1500°F for a period of two minutes, while the hottest temperature recorded on the structural steel members was in the order of 440°F.

A series of fire tests in Switzerland dealt principally with fire detection devices and automatic fire extinguishing systems in enclosed parking garages only. Both sprinklered and non-sprinklered tests were conducted. It is worth noting that in some of these tests the use of automatic sprinklers caused fire spread and more serious smoke problems than occurred in the non-sprinklered tests.

Survey of Fire Experience

The second phase of the study involved finding out what damage, if any, was caused by fires that had occurred in existing automobile parking struc-

Automobile burning 42 minutes into the test.



tures. A survey was made to provide a useful basis for determining the frequency of fires and extent of damage and dollar loss from fires in passenger automobile parking structures. The study was conducted with the close cooperation of the National Parking Association and the International Municipal Parking Congress. The results, based on a 46 percent return, covered 1,686 parking structures, having a total of 778,000 car spaces, in which there were 395 fires reported over a period of 60 years. A total of 368 of the 395 fires involved fire damage to the automobile only (one-third of these fires occurred over the last decade). Twenty-seven fires resulted in building damage, but did not involve replacement of beams, columns, or floors in the structure, and only four fires involved a loss exceeding $5,000. Sprinklers were not reported as a factor in the control of any fires. In all the fires reported, there was no injury or loss of life.

Automobiles were found to be the primary source of fire in the structure, but automobile damage was not a major loss category. In 77 percent of the fires, automobile damage amounted to less than $500.

The single most significant fact from the survey is that fire losses on the approximately $2-billion worth of real estate studied amounted to approximately $130,000 over a 60 year period.

The results of the survey justify a reevaluation of existing construction and fire protection requirements for modern parking structures. Further, the survey and the test data identify parking structures as a fire risk separate from all other types of occupancies.

The study of actual fires shows that automobiles act as containers and have very low fuel loads. If calculated on the same basis used to determine fire loads in other occupancies, the fire load in a parking structure is 2 lbs/sq ft of total floor area; in office and apartment buildings, it is usually 5 to 10 lbs/sq ft. (See table, next page.)

AISI Fire Test

To confirm all the data collected from fire tests in other countries and the findings in the survey of fire experience in automobile parking structures, the final phase, a full scale burnout test was planned for a modern, open-air steel frame parking structure, for the first time in the United States.

The test was conducted on October 15, 1972, in Scranton, Pennsylvania, in a modern, operating, multi-story open-air parking structure, with part of the ground floor level occupied by retail stores. Completed in 1970, the building has an exposed structural steel frame, with post-tensioned concrete decks. All structural steel in the parking decks is exposed.

Description of Test Site: Steel columns at the periphery of the decks support 54-ft long girders, spaced 18 ft o. c. The girders are coverplated W24X76 steel members, composite with a post-tensioned 5-in. concrete slab. The distance from one deck to the next is 10 ft, with 7 ft-7 in. clearance below the exposed steel girders.

During the test, cars were parked on the deck above the test bays, to provide normal live load on the girder spanning the test area.

Three automobiles were positioned, front first, into adjacent parking stalls, and spaced approximately 2 ft apart, as would normally be expected under actual conditions. In this position, the center car was located diagonally under an exposed girder, with its front in one bay and rear in another. The other two cars extended partially into other bays. The center car was then ignited.

Instrumentation: Temperatures in the test area were obtained from 40 thermocouples and recorded on charts at one 16-point and one 24-point temperature recorder. Placement of the thermocouples and recording of temperatures were accomplished by Underwriters' Laboratories, Inc. personnel.

Deflection of the steel girder spanning across the test site was measured by a surveying instrument, which was set up on the roof of the structure.

Exterior view of Scranton Parking Facility, site of full scale burnout test.

TYPICAL FUEL LOAD PER PASSENGER CAR

Type	Quantity (lbs.)	Calorific Values Btu x 10³	Combustible fuel Btu x 10³	Equivalent weight of wood (lbs.)
Fuel	85	20	17	212
Tires	101.2	17	17.2	215
Oil	13.8	19	2.6	32.9
Paint	18	10	1.8	22.5
Underseal	22.5	15	3.4	42.7
Cotton	14.9	9	1.3	16.5
Wood	14.1	8	1.1	14.1
Polyvinyl	34.9	10	3.4	40.5
Polyurethane	71.2	12	8.5	106.8
Totals	376#/car		56.3Btu/car	703#/car*

*Based on normal densities, this is equivalent to 2 lbs/sq ft of total floor area.

Horizontal expansion of the girder was measured by a plumb bob suspended from the column-girder connection and observed on a scale at the floor below.

The Scranton Fire Bureau had previously conducted a series of open-air burnout tests at their Fire Bureau Training Ground. Cars were set on fire under the engine compartment, in the interior of the cars, with full tanks and half tanks of gasoline, with the windows up and the windows down. The purpose of these tests was to determine what conditions would most likely contribute to a major automobile fire.

In the AISI burnout test, crumpled newspapers were added to the normal fuel load of the automobile (upholstery, lining, padding, carpeting, wiring, tires, oil and grease, etc.) and 10 gallons of gasoline to the fuel tank. (Half-full tanks were chosen as representing the worst condition. Tanks completely full represent less of an explosion hazard, and nearly empty tanks represent less of a spill hazard. In addition, a half-full tank comes closer to actual conditions in a parking structure.)

Conclusions: During 48 minutes of uncontrolled burning, the fire completely gutted the test car. From front to back all combustible materials in the automobile were consumed, with the exception of two quarts of gasoline (there were 10 gallons at the start of the test). There was no leakage at the seams of the gas tank. Later examination of the tank revealed no ruptures.

There was no structural damage. The only maintenance needed, following the test, was to steam clean the underside of the concrete deck to remove soot deposits, and to repaint the exposed steel girder directly over the test car.

The girder directly over the test car received more fire exposure than any other structural elements. The maximum temperature recorded on the bottom flange, located over the gas tank, reached 440°F at 41 minutes and lasted about five minutes. At 17 minutes after the test started, and lasting about two minutes, a temperature of 420°F was attained on the bottom flange over the car windshield, as flames flared out of the passenger compartment. For the most part, temperatures on the girder remained well below 350°F. This is significant when considering the limiting temperatures for such structural elements are 1100°F average, and 1300°F at any one point, as established by the American Society for Testing and Materials (ASTM-E119, Standard Methods of Fire Tests for Building Construction and Materials).

For a short time, at 42 minutes into the fire, as gasoline under the test car was burning vigorously, the deflection at the center of the 54-ft long steel girder reached 1⅝-in., and its horizontal expansion measured ⅛-in. These readings resulted entirely from thermal expansion of the bottom flange. After cooling, following the test, the structural element returned to its pretest condition, with deflection and elongation recorded at zero.

Air temperatures, measured by bare thermocouples suspended 24 in. below the slab, reached brief peaks as the fire flared up and then rapidly subsided. One thermocouple reached 780°F momentarily, at nine minutes after the start of the test; another peaked at 810°F over the windshield at 11 minutes. For the most part, the thermocouples recorded air temperatures well below 400°F because of the constantly moving flame, typical in automobile fires.

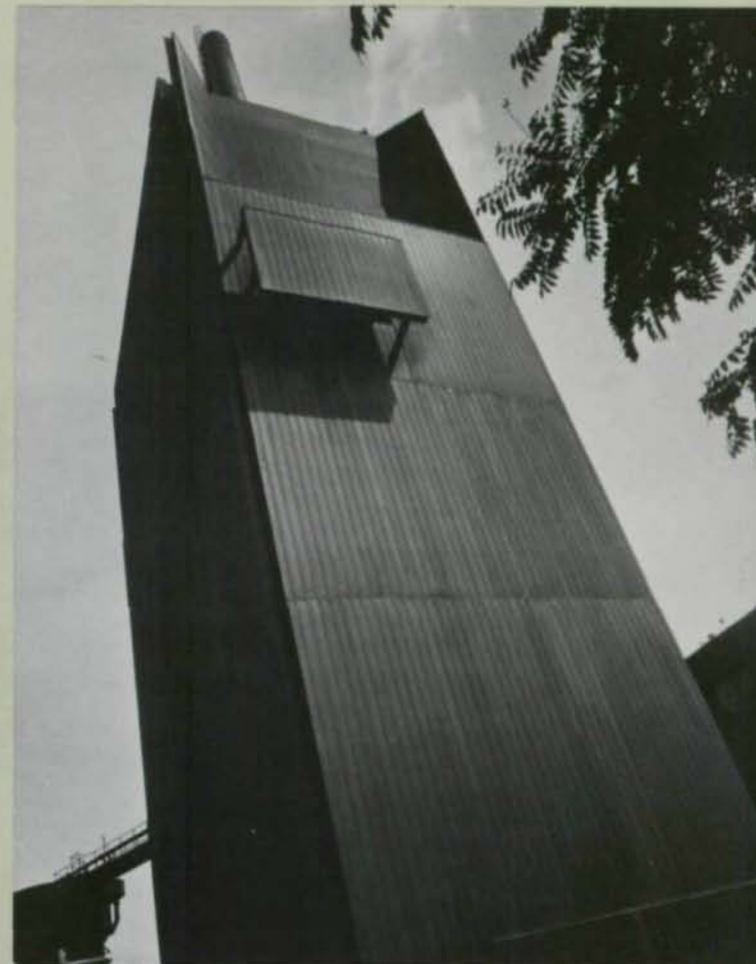
Fire exposure to the exposed steel structural members was never a critical factor during the test. Maximum temperatures and deflection of the girder directly over the fire remained well below significant levels. There was no damage to the exposed steel.

On the basis of this and other tests which have been conducted, it was concluded that open-air automobile parking structures represent an extremely low fire hazard and that exposed steel framing provides an adequate degree of fire safety against structural collapse.

The AISI burnout test is reported in detail in an engineering report by Gage-Babcock and Associates, Inc., available from the Engineering Division, American Iron and Steel Institute, 150 E. 42nd Street, New York, New York. A 26-minute film report has also been completed for selected showing to interested code, insurance, engineering, and architectural groups.

1972 Architectural Awards of Excellence

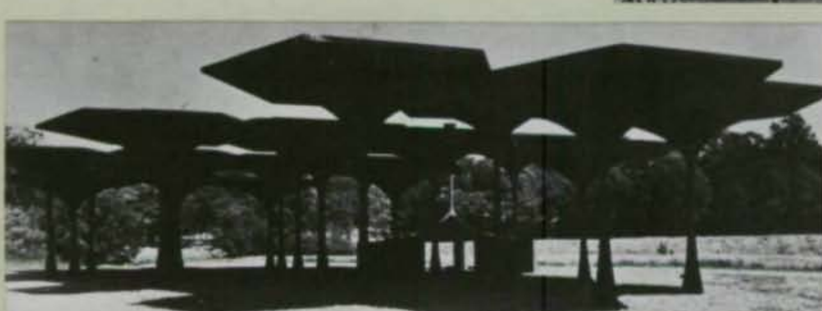
▲ **AIRCRAFT ACCESSORIES OVERHAUL SHOP (P-108) (P-110)**
San Diego, California
Architect: Daniel, Mann, Johnson, & Mendenhall



▲ **BOILER PLANT ADDITION**
Terre Haute, Indiana
Architect: Archonics Corporation

▲ **MCCORMICK PLACE ON-THE-LAKE**
Chicago, Illinois
Architect: C. F. Murphy Associates

PYRAMID BRANCH OF PEOPLES TRUST BANK ▶
Fort Wayne, Indiana
Architects: George Nelson and Gordon Chadwick
(Alan R. Grinsfelder, Supervising Architect)



◀ **COOK FIELD**
Yonkers, New York
Architect: Joseph Roth & Associates

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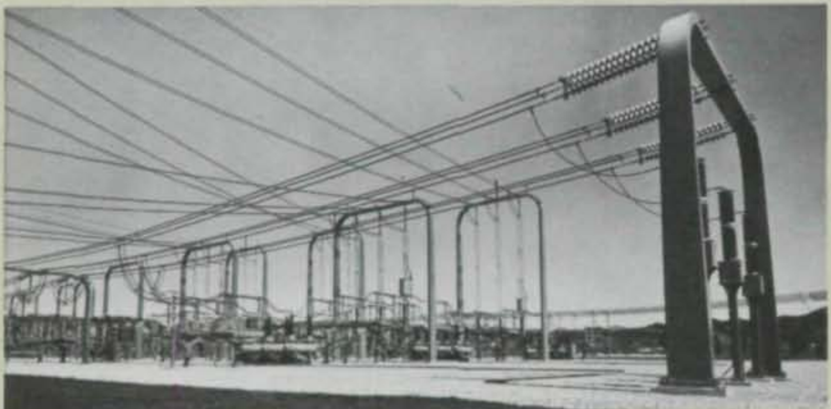

PARDEE SUBSTATION
Valencia, California
Architect: Southern California Edison Company