VOLUME XV / NUMBER 4 / FOURTH QUARTER 1975

MODERN STEEL CONSTRUCTION

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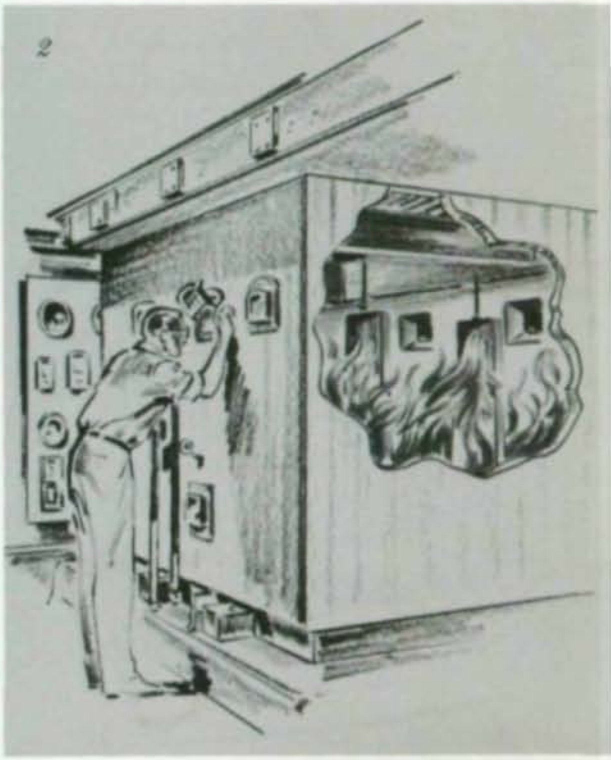

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MODERN STEEL CONSTRUCTION

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CONTENTS

NEW AISC QUALITY CERTIFICATION PROGRAM

In recent years, the quality of construction methods and materials has become the subject of increasing concern to building officials and designers. One result of this concern has been the enactment of inspection requirements intended to ensure product quality. However, in the case of fabricated structural steel these inspection requirements have increased fabrication costs without necessarily assuring fabrication quality.

Product inspection, although it has a valid place in the construction process, is not the most logical or practical way to assure that structural steelwork will conform to the requirements of contract documents and satisfy the intended end use. A better solution can be found in the quality control and quality assurance exercised by the fabricator throughout the production process. However, some valid and objective method is needed whereby a fabricator's capability for assuring quality production can be evaluated.

Recognizing this need, AISC has developed a Quality Certification Program whereby any structural steel fabricating plant —whether a Member of AISC or not—can have its capability for assuring quality production evaluated on a fair and impartial basis. Inspection-Evaluation of a plant, in accordance with standard requirements established by AISC, will be performed by an independent professional firm specializing in quality assurance.

This program was developed by a group of highly qualified operations personnel from large, medium, and small fabricating plants. It was reviewed and strongly endorsed by an independent Board of Review comprised of 17 prominent structural engineers from throughout the United States.

ABS Worldwide Technical Services, Inc. (a subsidiary of American Bureau of Shipping) has been retained by AISC to perform the plant Inspection-Evaluation. In the future, dependent upon demand, other independent professional firms may be involved.

For further information on the criteria, procedures, and implementation of this program, please write to AISC, 1221 Ave. of the Americas, New York, N.Y. 10020.

ERRATA

On the back cover of the 3rd Q., 1975 issue of MSC, the captions identifying the photographs of the City Avenue Pedestrian Bridge, Philadelphia, Pa. and the Madeira Beach Pedestrian Overpass, Madeira Beach, Fla. were transposed inadvertently. Both bridges were Award of Merit winners in the Special Purpose category in AISC's 1975 Prize Bridge Competition.

MARKET SQUARE ARENA

by Arthur W. McKinney

Beginning in the late 1960's, the United States experienced a tremendous increase in a highly specialized class of construction relating to large, indoor, multipurpose sports stadiums. During the same period of time, city governments across the country were seeking ways to revitalize decaying central business districts, and many cities considered the construction of indoor sports facilities as magnets to attract people and revenue into their central business districts.

Over the relatively brief period of six to eight years, many facilities were constructed, such as: the new Madison Square Garden in New York City; the Hampton Coliseum in Hampton Roads, Virginia; Scope in Norfolk, Virginia; the Richmond Coliseum in Richmond, Virginia; the Forum in Los Angeles; and many other smaller facilities. In the spring of 1971, the City of Indianapolis determined that such a facility would be highly beneficial and would integrate well into the existing Master Plan for revitalization of the central business district. The summer of 1971 saw the opening of the new convention center and, in the succeeding four-year period, the beginning of seven major office complexes within the city core.

The stadium's location was selected to bracket the central business district, triangulating with the new convention center and the Indiana University-Purdue University campus. The site consisted of two half city blocks bisected by a five-lane thoroughfare, Market Street, which would pass under the stadium floor and remain a main access way to the central business district. A traffic analysis of a ten block area surrounding the site, conducted

Mr. McKinney, of J. Robert Carlton & Assoc., Richmond, Va., was Chief Structural Engineer for the Market Square Arena.

by the Indiana Department of Transportation, revealed no significant problems at peak traffic periods.

An analysis of parking requirements for an 18,000-seat facility was made. Seven thousand five hundred commercial parking spaces exist to the west of the stadium site within a five-minute walk. It was determined that an additional 1,400 spaces would be provided within the building. Economic estimates, made at the time, indicated a potential draw of two million people into the downtown area annually for stadium events. This provided the impetus for the city to proceed with the project.

Complex Ownership

Ownership of the building is unique. The land for the project was purchased and all existing structures demolished by the city. A private investment group provided financing for the parking decks and holds ownership and operates the parking decks. The city, through general obligation bonds and revenue sharing capital, provided funding for construction of the stadium and street replacement and improvements necessitated by the construction. The private investment group leases the stadium from the city, thus generating the funds for repayment of the bonds. The private investment group further obligated itself to construction of a high-rise office building adjacent to the site, and this project proceeded separately and will be completed this year.

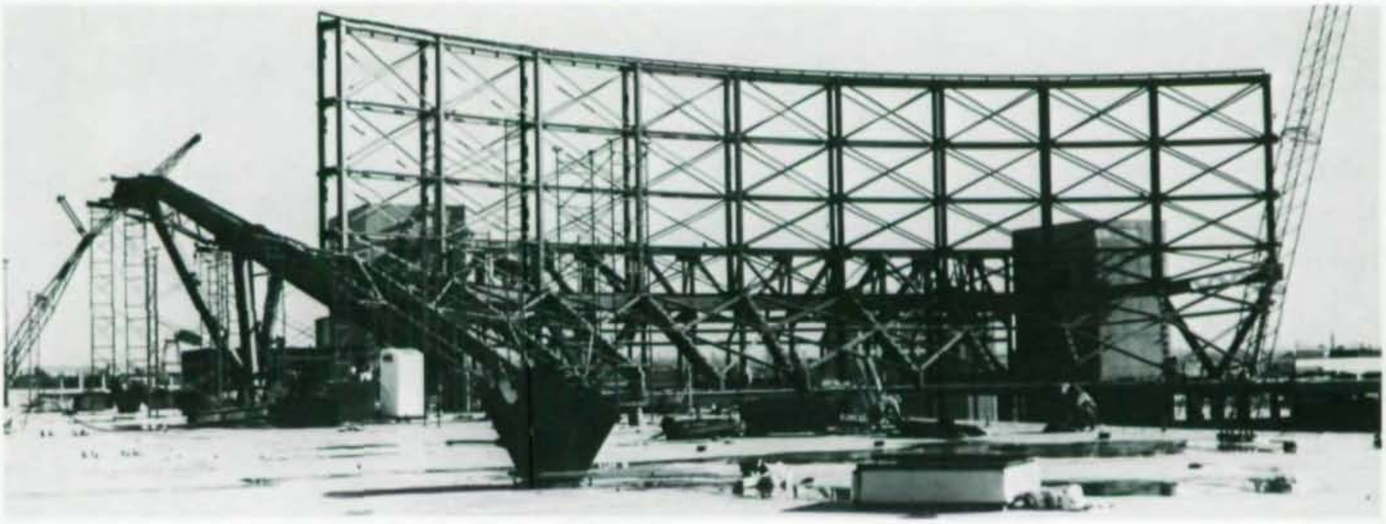
In May 1971, the City of Indianapolis selected the firm to provide professional engineering and architectural services. Schematics for the project were completed in August 1971, and a contract for demolition and site clearing was awarded in that same month. During the schematic design phase, it was determined that the most economical approach to constructing the facility would be the use of construction management and the fast-track phased construction technique.

"Fast Track" Technique

A construction management firm was selected on the basis of professional competence in much the same manner of architect and engineer selection. This was one of the first incidences of a public body entering into pure construction management, phased construction. This technique accrued many advantages in the area of cost control and time savings. Eventually, a total of 29 prime contracts were let. The contracts were all publicly bid and each contract was direct with the city and administered by the construction manager acting as the city's agent.

Using this technique, construction follows the design very closely. In October the contract for pressure injected piles was bid. This particular system was selected due to the existence of a dense gravel and sand strata at a depth of only 12 ft below grade. Conventional belled caissons and spread footings were considered. However, due to the large column loads imposed (as high as 2,400 kips), it was decided to use piles. Each pile had a load capacity of 150 tons and an average length of approximately 12 ft. A total of 889 piles were eventually required. Design

Market Street, a 5-lane thoroughfare, passes under the stadium floor.



Space frame and upper stadium wall erection, east end.

of the pile foundation preceded installation of the piles by one day for a period of approximately a month until symmetry could be brought to bear. It should be noted that the design of a building normally proceeds from the top down, while construction proceeds from the bottom up. This presents a dichotomy when both tasks are being accomplished simultaneously. Utilization of the "fast-track" technique therefore requires significant coordination and forethought.

Space frame of north garage.

Design Criteria

For design purposes the project was split into two distinct elements—first, the stadium proper and its associated ancillary facilities and, secondly, the split parking decks for the stadium. The design criteria called for professional basketball and hockey events with minimum seating capacities of 18,000 and 16,000, respectively. These two events account for the majority of the dates for the facility and dictate the bulk of the secondary design requirements. The single focal point line of sight is the most important requirement for these events.

For basketball the common focal point is normally the edge of the court playing surface. For hockey it is a point 2 ft above the playing surface and 6 ft in front of the dasher boards. To achieve a common focal point, the seating must follow a curve of decreasing radius with distance from the focal point. This curve can be approximated by a logarithmic spiral, by a function of a hyperbolic cosine, or by some logic approach used by many seating companies of maintaining a 3-in. differential between the eye level of the spectator in row 3 over the head of the spectator in row 1, and so on. The common focal point requirement established the first criteria for the structural geometry. Other requirements for the stadium were that it be provided with all the necessary facilities to accommodate circuses, sideshows, concerts, and similar events requiring specialized anchor devices on both the floor and roof systems and also requiring that the stadium space be acoustically treated. Further requirements were lighting levels for color television coverage of events (250 ft-candles on any horizontal surface on the playing floor). The stadium was to be fully air-conditioned, have humidity control, and a complete life safety protection package. Through economic analysis of all existing stadiums of similar size and all systems proposed for clear spans in the range of 300 to 400 ft, it was determined that the stadium roof configuration would be circular and would utilize a compression shell. This decision set the second parameter for the structural geometry.

Efficient Seating Arrangement

Due to economics, it was determined that the floor area would be the exact dimensions of the hockey playing floor required by the National Hockey League. The most efficient arrangement of seating layout would then be concentric expansion of this configuration until the required number of seats was reached. The NHL playing floor is 85 ft wide, 200 ft long, with a 28-ft radius in the four corners. The minimum row width (using self-rising seat bottoms) is 32 in. Expanding concentrically from the hockey floor on a 32-in. module, with appropriate deductions for vertical and horizontal aisles, a dia. of 368 ft-4 in. was determined.

This particular approach causes the number of rows to vary due to the variations in distance from the edge of the playing floor to the circular inner wall of the stadium. This approach further defined the necessary structural geometry, since the seating is parallel to the straight sides of the hockey floor and forms concentric circular arcs off of the center point of the 28-ft radius in the playing floor corners with the aforementioned geometric restraints. Economic analysis indicated that use of steel stringers supporting precast, prestressed seat risers would provide the most economical structural system for the seating. Due to the fast-track technique being utilized, it was determined that the roof would be supported at 48 equally placed points at its outer periphery and that the seat stringers coming up from the stadium floor level would intersect these 48 points.

Preliminary analysis of the seat stringers indicated that on the north and south sides of the stadium the required steel section would exceed the largest rolled section in the United States (W36X300). Therefore, intermediate support points were selected for the north and south side stringers to allow the use of the W36X300. This produced, including the 48 common dome/seat stringer support points, a total of 76 points at which the upper stadium structure required support.

Two Parking Garages

Nested against the stadium under the concourse level and covering the entire half city block on each side of Market Street are two 5-level parking garages. The stadium floor, which spans the street, coincides with the third level of parking.

The required 76 support points for the dome had to be transferred to the foundation through the garages without compromising the parking density or traffic patterns in the garages. This was accomplished by developing a fully triangulating space frame, the basic element of which is a four-joint column cluster. The two outer columns are splayed and each carries one of 48 common dome/seat stringer support points. The inner pair of columns is splayed and provides intermediate support to the upper seat stringer section to keep this section from exceeding the maximum size manufactured. To complete the triangulation of the space frame, the sixth floor was reinforced to provide the bottom chord. Thus, the space frame consists of the splayed cluster columns, the seat stringer, and the concrete sixth floor deck.

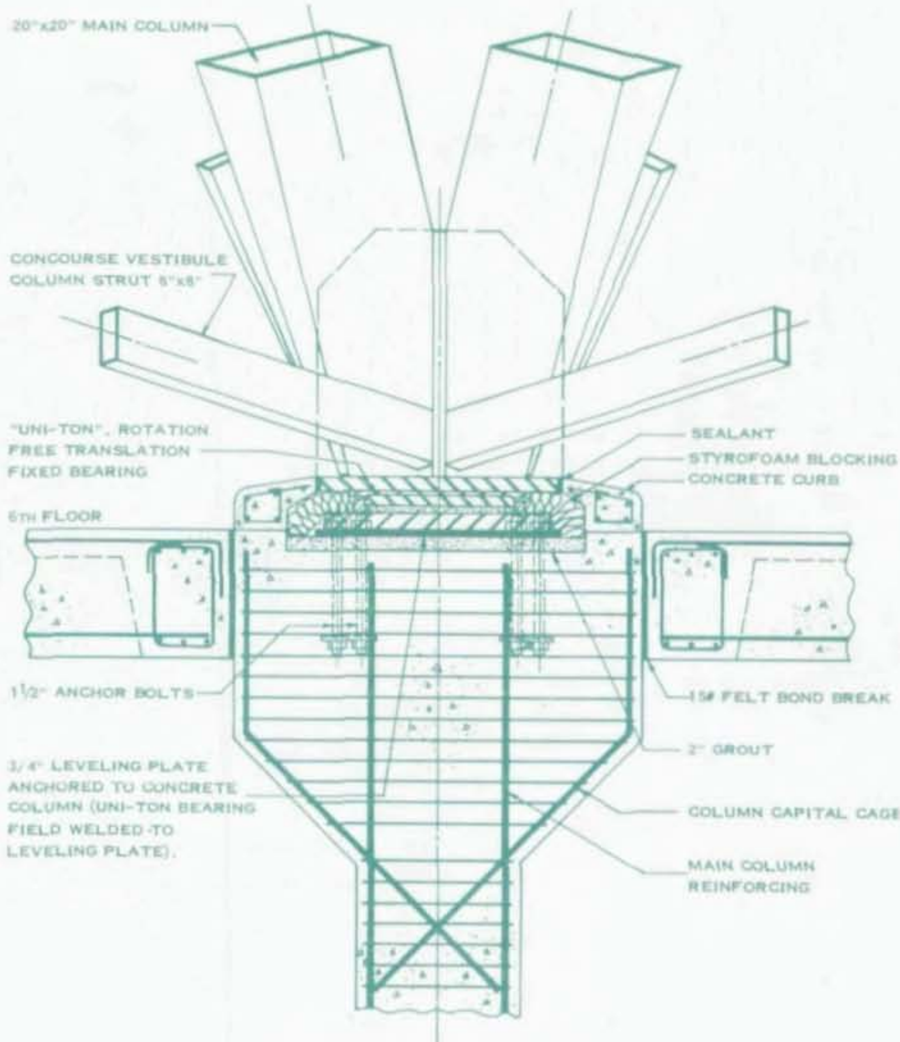
Thermal Gradients Consideration

The final criteria that were applied involved provision for expansion and contraction due to thermal gradients. The steel structure spanning the street is not connected to the garages or the stair towers. The third floor is cut with expansion joints running north and south just beyond the ends of the hockey playing surface. The space frame is cut, up to the arena wall, at the intersection of the garages and the steel structure spanning the street. This was accomplished by simply leaving out the diagonal bracing at the appropriate location. It is similarly cut at the center line of the garages where the expansion joint exists in the garages. The upper stadium wall is completely continuous and the dome is completely continuous. To analyze the behavior of the structure under thermal gradients, including displacement in the space frame induced by movement of the garages, required computer analysis due to the extreme complexity of the system. The space frame was modeled and analysis was performed using STRUDL II of the ICES package (Integrated Civil Engineering Systems—Structural Design Language). Two iterations were required to arrive at the final configuration, sizes and joint design criteria. Analysis indicated that the space frame behaves in a mode which is essentially the reverse of a compression shell.

The critical element of stability is the upper stadium wall. A full cross-section through the entire structure is shown on pg. 9. It is cut at the building center line looking west. The dome is an articulated compression shell commonly referred to as a Schwedler dome. Suspended from the dome is a steel catwalk system which carries the lighting system and provides access to the sound system and scoreboard.

Structural Steel

The exposed columns and cluster bases of the space frame are fabricated of weathering steel. Clip angles were provided for positioning the column tubes on to each element of the cluster base. Once the tube was in position, the column tube was welded to the cluster base with a full penetration weld using B-80 electrodes. After



Detail at cluster column base.

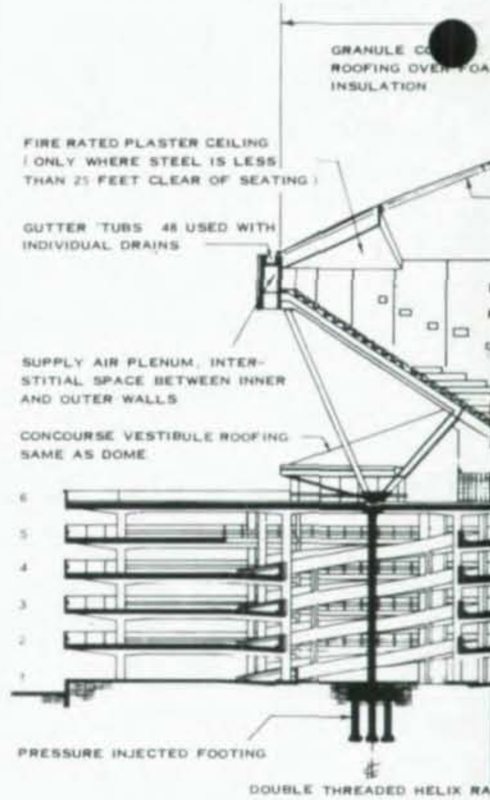
Typical four column "cluster" base erected with bearing in place.

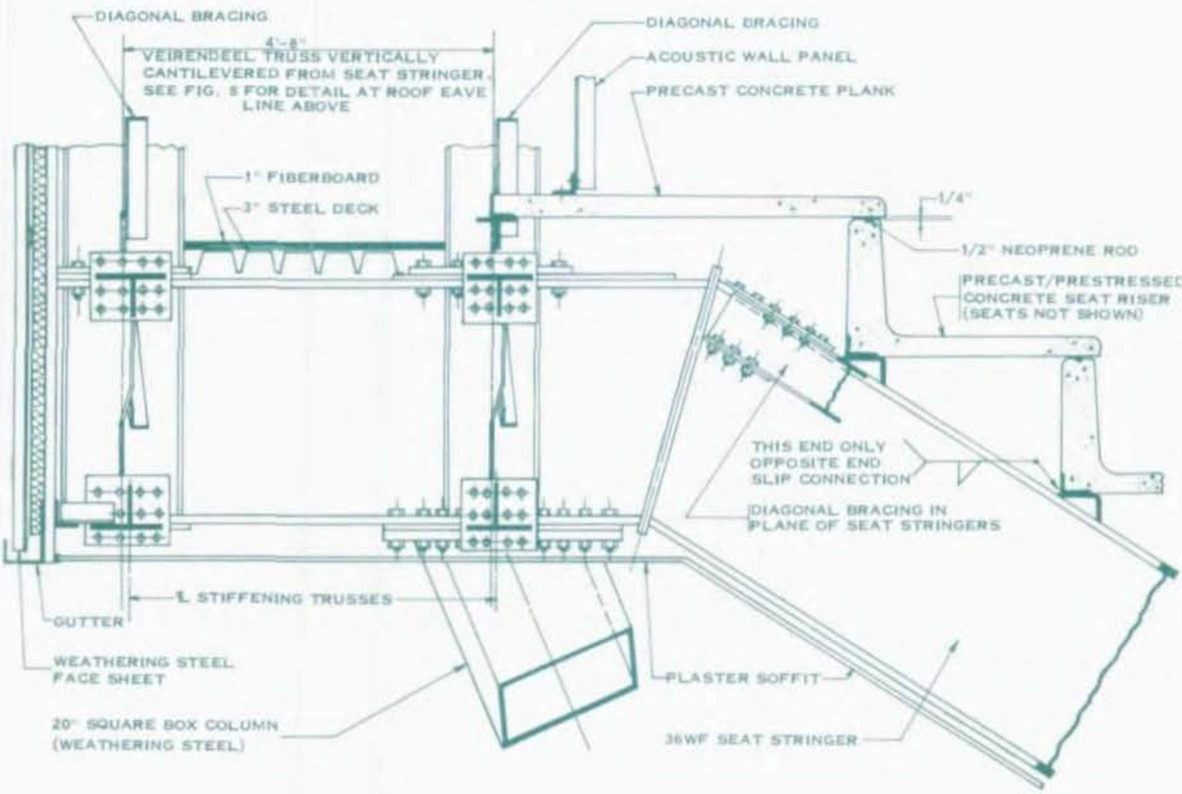
welding, the clip angles were burned off and the surfaces ground flush and sandblasted to insure uniform weathering of the steel.

The column bases were installed with a "uni-ton" rotation free, translation fixed bearing attached to the base with temporary pins to maintain alignment. These pins were burned off after the completion of erection. To allow for field tolerances, the "uni-ton" bearing base plate was field welded to a preset leveling plate. Similar detailing was utilized where the cluster bases rest on the steel structure over the street. The column tubes are 20 in. square with wall thicknesses varying from ¾ in. to ½ in. Prior to fabrication of the column bases, a full scale mock-up was constructed and then cut apart for destructive tests to qualify the welds. This also provided a test of the welding sequence to minimize plate wearing during fabrication.

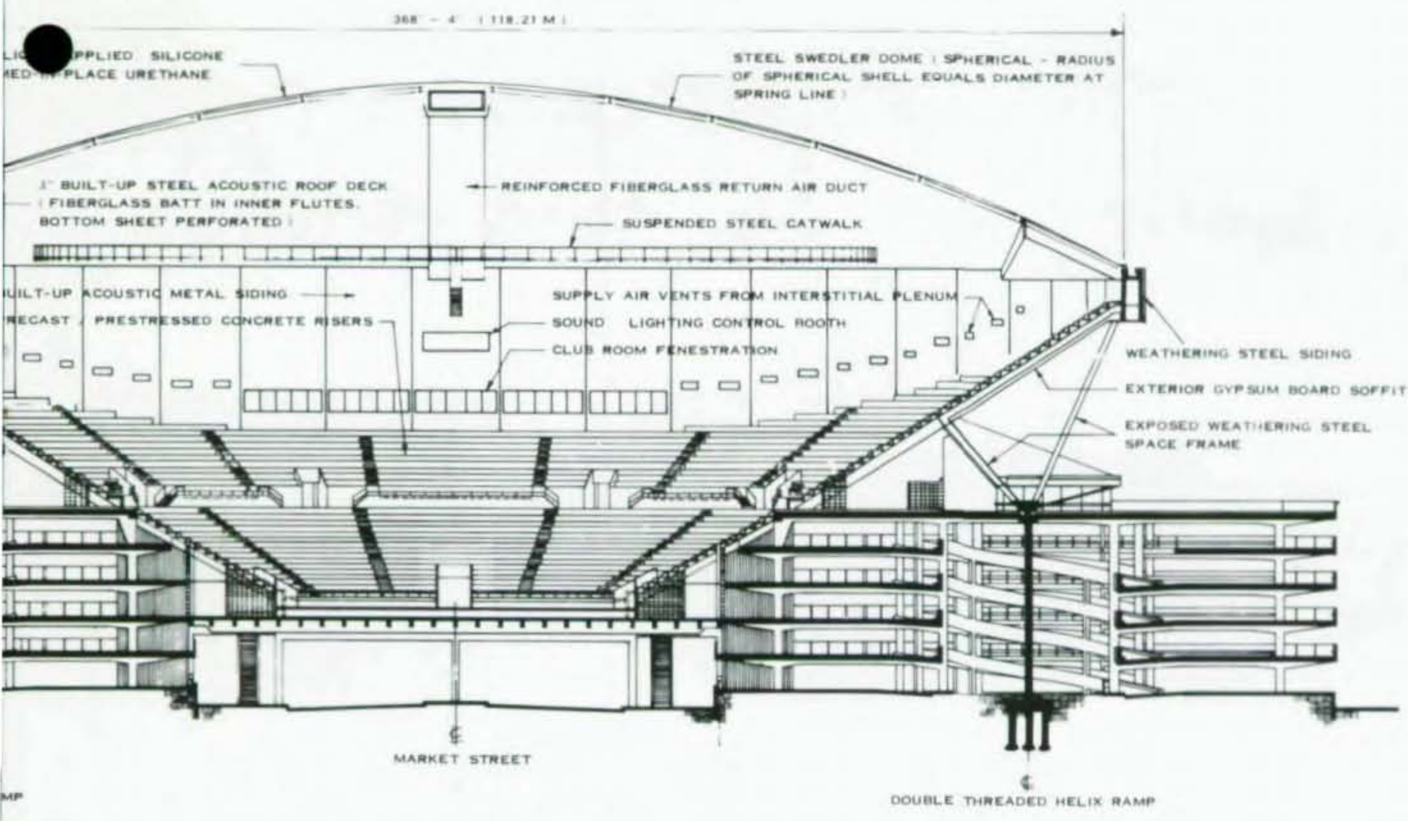
Detailing of the stringer head was complicated in that the stringer was broken into a horizontal plane and rotated in plan to fall on a radius to the center of the dome. The upper wall framing in the plane of the seat stringer, which has been turned so that it lays in a common plane with the dome rib, was treated as a vertical cantilevered vierendeel truss. The inner chord of this truss supports the dome rib. The dome is tangentially restrained, but is free to move radially. The major horizontal forces induced in the upper wall come from radial movement of the dome due to temperature changes.

The force is alleviated by providing Teflon sheets with a very low coefficient between the dome ring bearing plate and the column cap plate.





Seat stringer head at outer arena wall.



Cross-section at center line looking west.

The dome ring consists of a single steel plate 36 in. deep and 4½ in. thick. Shop splices in this plate were made using the electro-slag welding process. Field splices consist of six 3-in. diameter bolts connecting adjacent segments. The ends of the segments have a gusseted bearing plate similar to the type of detail that may be used for a heavy column base connection. The connecting bolts were post-tensioned to 1,800 kips to prevent separation of the connection under the most severe tension in the ring. The electro-slag welding technique was also used in the construction of the dome ribs.

The ribs are broken on approximately 28-ft centers to approximate a true spherical curve. At the breakpoints, the bottom flange was cut away, the web was notched to the upper flange line. With the rib lying on its side, the upper flange was bent by use of hydraulic jacks. This closed the notch in the web which was then welded and left approximately a ⅝-in. gap in the bottom flange which was closed using the electro-slag welding process.

The dome was erected in two stages using a system of shoring trusses. The lower portion of the dome, approximately the first 96 feet of the span, was set in place with the inner ends of the ribs resting on steel chairs on the shoring system. The annular rings were then installed along with the secondary purlins and 2-in diameter rod bracing. The shoring was then dropped 24 in. A center tower was erected with the compression ring mounted to the top of the tower. The balance of the dome was then erected and the shoring dropped. Use of two swings to drop the dome into final position provided better field control during erection.

Architects: Architects 4 (A Joint Venture)
J. Robert Carlton & Associates, Inc.
Richmond, Va.
Kennedy, Brown & Associates
Indianapolis, Ind.
Fleck, Burkart, Shropshire, Boots, Reid & Associates
Indianapolis, Ind.
McGuire & Shook Corporation
Indianapolis, Ind.

Structural Engineer:
J. Robert Carlton & Associates
Richmond, Va.

Construction Manager:
Huber, Hunt & Nichols, Inc.
Indianapolis, Ind.

Steel Fabricators:
International Steel Company
Indianapolis, Ind.
American Bridge Div.,
United States Steel
Pittsburgh, Pa.

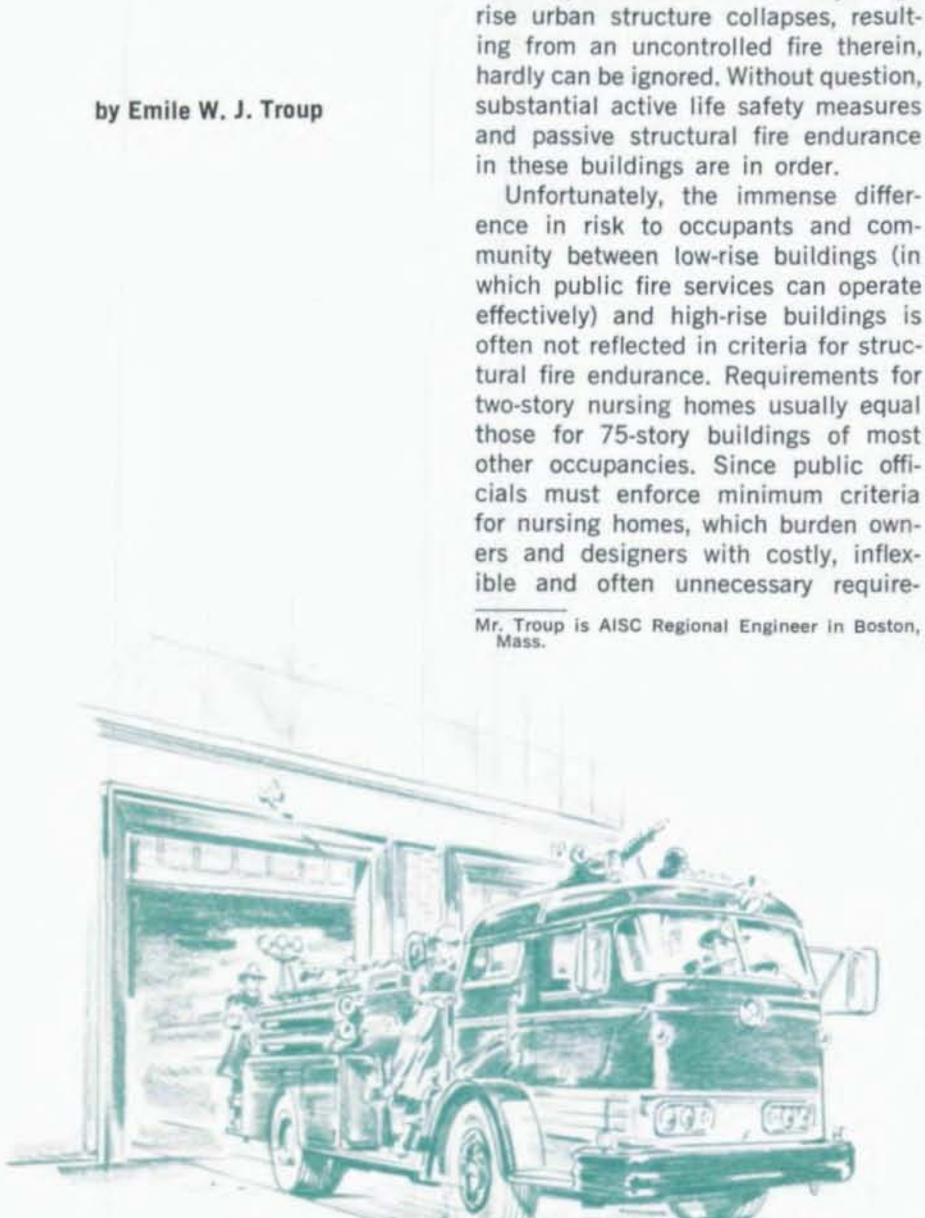
FIRE ENDURANCE FOR
Steel-Frame Nursing Homes

by Emile W. J. Troup

In recent years, much attention rightly has been given to fire safety in high rise buildings. The possibilities of catastrophic loss of life and major high rise urban structure collapses, resulting from an uncontrolled fire therein, hardly can be ignored. Without question, substantial active life safety measures and passive structural fire endurance in these buildings are in order.

Unfortunately, the immense difference in risk to occupants and community between low-rise buildings (in which public fire services can operate effectively) and high-rise buildings is often not reflected in criteria for structural fire endurance. Requirements for two-story nursing homes usually equal those for 75-story buildings of most other occupancies. Since public officials must enforce minimum criteria for nursing homes, which burden owners and designers with costly, inflexible and often unnecessary requirements for fire endurance of the structural frame, it is important for architects, structural engineers, and code enforcement officials to learn of the procedures now available for design of fire-resistant steel frames.

Mr. Troup is AISC Regional Engineer in Boston, Mass.

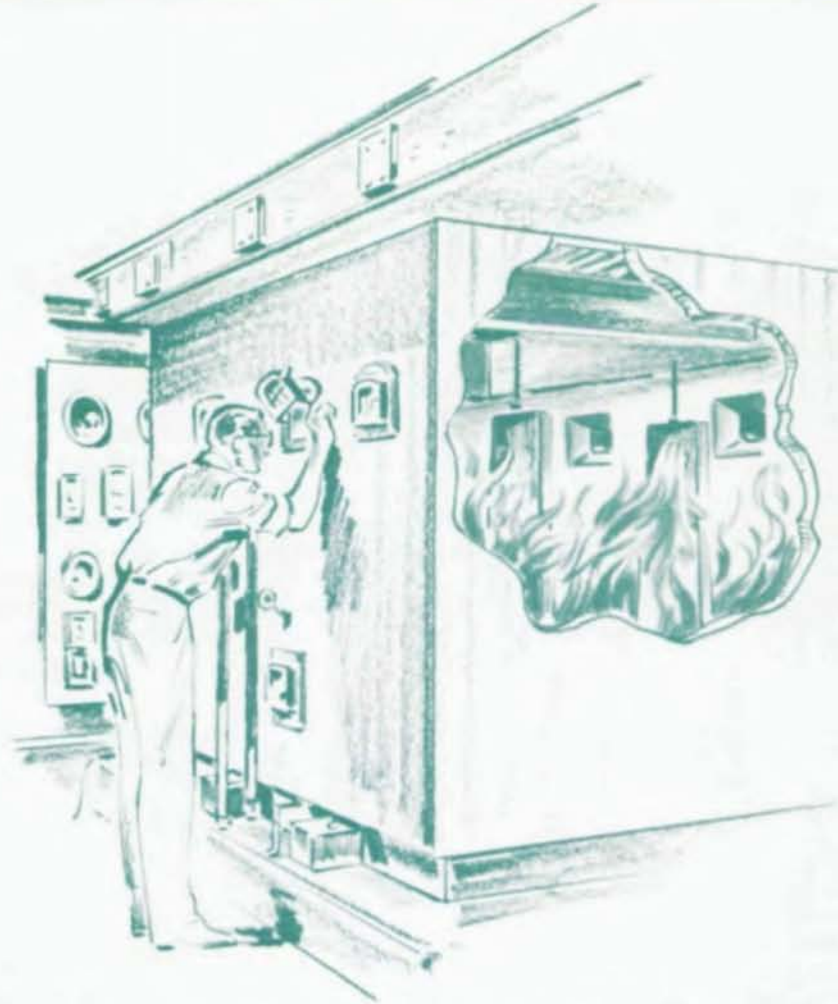


Design Criteria

Nursing home design, requiring both institutional and residential occupancy, is a difficult and unique challenge. Architects must provide a pleasant, homelike environment for the residents that complies with the strictly regulated state and federal fire safety design laws. For example, to conform with limits on distance to the most remote room, large nursing homes are often laid out in units or wings radiating from a central nurses' station.

In general, there are no unusual features of nursing homes which require unique structural treatment. However, like other institutional facilities, future expansion or alteration of a nursing home is inevitable, and often sooner than expected. Expansion is needed to increase capacity; alterations are necessary to bring an existing nursing home up to current safety standards, since the "grandfather clause" is not valid. Indeed, some nursing homes have ceased operation because of the prohibitive cost required to conform to recent construction fire safety regulations. In this regard, structural steel framing is clearly preferred because of its flexibility in adapting to future changes and expansion.

Most states, in accordance with a federal law, require all new nursing homes receiving Title 18 or 19 funding (Medicare or Medicaid) to comply with National Fire Protection Association 101, the **Life Safety Code,** 1967 edition. NFPA 101-67 has been revised



twice (with another edition due in 1976), and a few states apparently have begun, through legislative action, to recognize the latest edition (1973) as the guideline for life safety in nursing homes.

Under NFPA 101-67, a new two-story nursing home must have, **as a minimum,** 2-hour fire resistive construction (Type 1B). Through NFPA 220, **Standard Types of Building Construction,** 1961 edition, Type 1B construction specifies 3-hour rated columns for the first floor, 2-hour columns for the second floor, and a 2-hour floor system. These ratings do not recognize complete sprinkler protection or the other life safety measures which are usually provided.

There is no justification for the additional expense to obtain this degree of fire endurance for structural members in modern nursing homes, which are usually only two or three stories high. Although some difficulty should be anticipated in guiding occupants out of the building during a fire, several hours of structural fire endurance could not conceivably have any bearing on life safety. This conclusion has been drawn in a recent study of nursing home fires in Oregon.[1] And for insurance purposes, complete automatic sprinkler protection in a nursing home of unprotected noncombustible steel frame construction results in classification as a fire-resistive building.

A more reasonable criterion for the frame would be minimum fire endurance that would insure structural integrity during the time needed for egress and rescue of the occupants. Depending on such factors as public fire protection, Type 2B or 2C construction (protected or unprotected steel frame, respectively) would usually be adequate. New nursing homes 5 or 6 stories or higher, which are rare, might warrant a Type 2A construction (1½ and 2-hour columns, 1½-hour floor systems), since the life safety problem therein is somewhat more complicated.

It should be mentioned that, contrary to the "zero" rating listed in building codes, "unprotected" steel construction (Type 2C) does provide measurable fire endurance by its own capacity to conduct and store heat. Theoretically, a bare steel structure *can be designed heavy enough to meet* any fire resistance rating. The author does not, however, submit this as a practical alternative to fireproofing with insulation, at least not for the general case. The point is that Type 2C construction, in combination with life safety measures, provides a very fire-safe and economical low-rise structure for many occupancies. Furthermore, Type 2C construction does not produce the primary killer, smoke, nor does it contribute fuel to the fire.

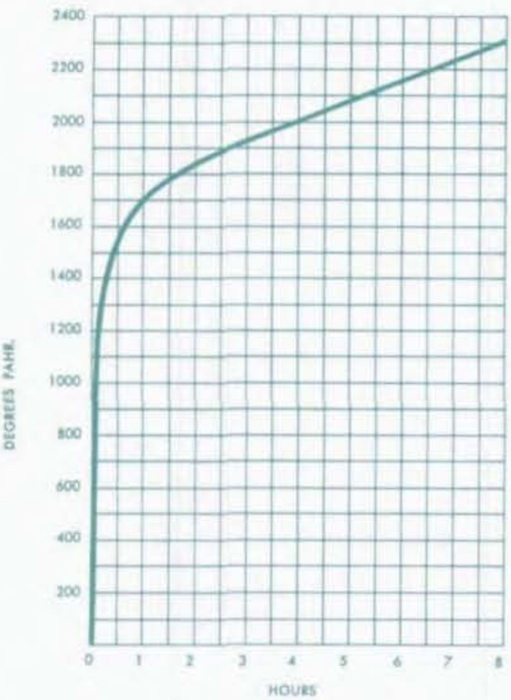
Fire Tests In Perspective

Along with the misconception that 2- and 3-hour structural fire endurance contributes to life safety, there is another difficulty in designing safe, economical, low-rise nursing homes. Until recently there was no ASTM E119 fire test data on light steel column assemblies using fabricated W or tubular shapes. In order to meet NFPA 101-67, designers found they either had to use heavier sections than needed (at least W10X49) or Lally-type column assemblies.

It is unreasonable to expect or require architects and structural engineers to specify only those column, *beam and floor assemblies which have* been subjected to the E119 fire test. It is more unreasonable to expect every variation of these assemblies to be so tested. On the contrary, results of fire tests are a valuable source of data from which engineering judgment should be made to assign a fire resistance rating to an assembly which is similar (but not identical) to those already tested. These results should be made available to aid designers and code enforcement officials in making this judgment.

The E119 test fire is a representation of actual fires in buildings. In some cases, and nursing homes are a good example, the E119 3-hour exposure is not indicative of an actual

3-hour fire which would be conceivable in the building. It is extremely unlikely, if not incredible, that fire temperatures at a specific location would average 1800°F for 2½ hours as specified by the standard 3-hour test fire. Indeed, the writer simply cannot conceive of this kind of exposure to first story columns in modern nursing homes.



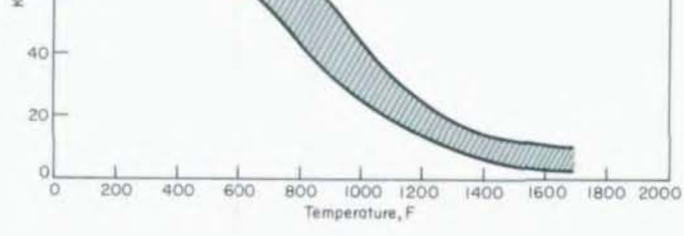
ASTM time-temperature curve shows that in the standard fire test, air temperature rises to 1000° F in only 5 minutes, reaches 1700° F at 1 hour, and 2000° F at 4 hours.

The E119 test is an elaborate index of relative performance under controlled laboratory test conditions for building constructions.[2] Since the E119 test conditions are never experienced in an actual building, analysis and interpretation of existing fire test results are necessary if rational design of the structural frame is to be achieved.

New Design Procedures

The ASTM E119 fire test method will continue to have an important role in development of fire rated steel-frame assemblies, especially in evaluation of new insulating materials and methods of assembly, and in the development and validation of analytical techniques. As an example, analysis of existing data supplemented by a fire test program recently resulted in procedures which, for the first time, enable the architect and engineer to truly design individually protected steel-frame assemblies.[3,4,5] Knowing the required hourly rating, required thickness of most insulations can now be calculated for any size or shape steel column. The designer is limited only to the use of **proven (fire tested) insulation materials and methods of assembly.** This breakthrough has been possible because of the knowledge now available of: 1) behavior of structural steel at high temperatures and 2) the performance of a wide variety of insulating materials and assemblies used as "fireproofing."

Tensile strength of ASTM A7 and A36 structural steels at elevated temperatures.



Below is an example of how these new procedures can be used.

From Ref. 1:

$$L = \frac{R}{\frac{20\ W}{DP} + C}$$

where L = thickness of insulation, in.
R = fire resistance, hours
W = weight of steel, lbs/ft
D = heated perimeter, in.
P = density of insulation, pcf
C = constant: 0.5 for materials such as sprayed fibers and dense mineral wool; 1.2 for materials containing portland cement or gypsum, such as plasters and cementitious mixtures.

Assume a W6X15.5 column, R = 2 hours (fire rating required).

For **plaster on metal lath,** C = 1.2; box-type protection, assume P = 33: NEED L = 1.27 in.; USE 1¼ in.

For **sprayed-on cementitious,** C = 1.2; contour protection, assume P = 25: NEED L = 1.30 in.; USE 1⅜ in.

For **dense mineral wool,** C = 0.5; box-type protection, assume P = 20: NEED L = 1.78 in.; USE 1¾ in.

For **sprayed-on fibers,** C = 0.5; contour protection, assume P = 15 (tamped): NEED L = 1.88 in.; USE 1⅞ in.

Although the formula above was developed from investigations of column fire tests, in many cases it may be used for individually protected beams and girders as well.

Sprinklers

The question of reliability of sprinkler systems is under constant and heated debate. Insurance interests, which have traditionally influenced fire protection requirements in buildings, have learned to live with the various failure modes of sprinkler systems in achieving acceptable levels of industrial (property) loss experience. However, the extreme risk to occupants and community from an uncontrolled fire warrant a very hard evaluation of the reliability of sprinklers in high-rise buildings. Failure of the system to function properly when needed in a high-rise could be catastrophic compared to one of the structural steel elements reaching 1100°F.

Life safety hardware and systems are unique utilities in a building. For several dollars per sq ft of floor area, the owner buys equipment most of which performs no day-to-day useful function and, in fact, will probably never be called upon to operate under hostile conditions. Maintaining this kind of equipment over a period of several decades presents perhaps the most challenging reliability problem. Some of the documented sprinkler failure modes which arise in industrial properties could also occur in a nursing home. Under some circumstances sprinklers can be outmatched by fires occurring in areas with high ceilings above 15 to 20 ft.[6] Fortunately, this condition is not to be found in nursing homes.

When all the arguments are sorted out, it is apparent that an operational sprinkler system can be a deterrent to **multiple** loss of life in a nursing home fire. Likewise, sprinklers further reduce chances of structural damage. Adequate and continuous sprinkler system reliability can be assured with a simple maintenance program.

Perhaps for these reasons, NFPA 101-73 allows a drop of one construction Type, from 1B to 2A, if complete sprinkler protection is provided in two and three-story nursing homes. This is a step in the right direction. Although the writer does not endorse this "trade-off" concept for primary structural members in high-rise buildings, **it should definitely be applied to low-rise structures.** Most new nursing homes however, cannot make use of this valid concept.

Summary

Certainly there are elements of building design and construction which can greatly influence the degree of life safety in modern nursing homes. Since nearly all fire injuries and fatalities in such structures result from small, single casualty fires,[1] it must be concluded that fire endurance of a structural steel frame is not a relevant factor. If a large loss of life should occur in a steel-framed nursing home fire, it will be a result of deficiencies of the other factors upon which life safety really depends.

The following recommendations are made:

1) Where applicable, use the new procedures now available for designing individually protected fire-resistant steel sections.
2) Insist due credit be given for complete sprinkler protection, if provided, in terms of anticipated fire exposure of noncombustible structural members.
3) Exploit existing fire test results to their best engineering advantage.
4) Minimize the potential threat to life safety by minimizing combustible furnishings and contents.
5) Provide early detection and alarm systems for life safety.

It would appear in this particular occupancy that adequate protection of the residents will, in effect, satisfy any structural fire protection consideration that may arise. But, until structural fire endurance is placed in its proper perspective, the measures described herein will be helpful in achieving rational structural design of nursing homes.

References

1. *Stickney, C. Walter* "Oregon Life Safety Study of Institutional Care Facilities" *Technology Report 74-3, Society of Fire Protection Engineers.*
2. *Gewain, Richard W.* "Fire-Resistant Steel Frame Construction" *Paper presented at the National Engineering Conference sponsored by The American Institute of Steel Construction, Inc., May 2, 1975.*
3. *Lie, T. T. and W. W. Stanzak* "Fire Resistance of Protected Steel Columns" *Engineering Journal, American Institute of Steel Construction, Vol. 10, No. 3, 1973.*
4. "Fire-Resistant Steel-Frame Construction" *American Iron and Steel Institute, Second Edition, 1973.*
5. "Designing Fire Protection for Steel Columns" *American Iron and Steel Institute, Second Edition, 1975.*
6. *Troup, E. W. J.* "New Developments in Ceiling-Level Protection for the High-Challenge Fire" *unpublished Technical Report, Factory Mutual Research Corporation, January 1974.*

1975 ARCHITECTURAL AWARDS OF EXCELLENCE

FODREA COMMUNITY SCHOOL
Columbus, Indiana
Architect: Caudill Rowlett Scott

IVAN G. SMITH ELEMENTARY SCHOOL
Danvers, Massachusetts
Architect: Caudill Rowlett Scott

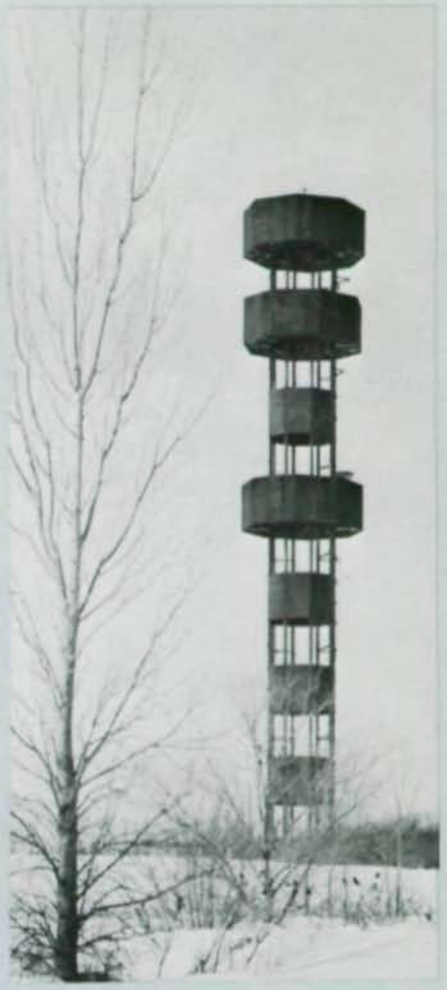
LECKENBY COMPANY OFFICE BUILDING
Harbor Island, Seattle, Washington
Architect: Ibsen Nelsen & Associates

ALLISON PARK RESEARCH CENTER
Hampton Township, Pennsylvania
Architect: Giffels Associates, Inc.

CROSBY KEMPER MEMORIAL ARENA
Kansas City, Missouri
Architect: C. F. Murphy Associates

▲ **BLUE CROSS AND BLUE SHIELD OF NORTH CAROLINA SERVICE CENTER**
Chapel Hill, North Carolina
Architect: Odell Associates Inc.

THE MINNEAPOLIS-PLYMOUTH RADIO TOWER
FOR THE NORTHWESTERN BELL TELEPHONE COMPANY ▶
Plymouth Village, Minnesota
Architect: Setter, Leach & Lindstrom Inc.



BAXTER CORPORATE HEADQUARTERS, CENTRAL FACILITIES BUILDING
Deerfield, Illinois
▼ Architect: Skidmore, Owings & Merrill

◀ **FIRST WISCONSIN CENTER**
Milwaukee, Wisconsin
Architect: Skidmore, Owings & Merrill

FOURTH FINANCIAL CENTER
Wichita, Kansas
Architect: Skidmore, Owings & Merrill ▼

▲ **SEARS TOWER**
Chicago, Illinois
Architect: Skidmore, Owings & Merrill

▲ **WESTINGHOUSE RESEARCH AND DEVELOPMENT CENTER: ADMINISTRATION BUILDING**
Churchill Borough, Pennsylvania
Architect: Skidmore, Owings & Merrill

VISITORS INFORMATION CENTER, TROJAN NUCLEAR POWER PLANT
Rainier, Oregon
▼ Architects: Wolff Zimmer Gunsul Frasca Partnership