VOLUME XIX NUMBERS 3 & 4 / THIRD & FOURTH QUARTER 1979
VOLUME XX NUMBER 1 / FIRST QUARTER 1980

MODERN STEEL CONSTRUCTION

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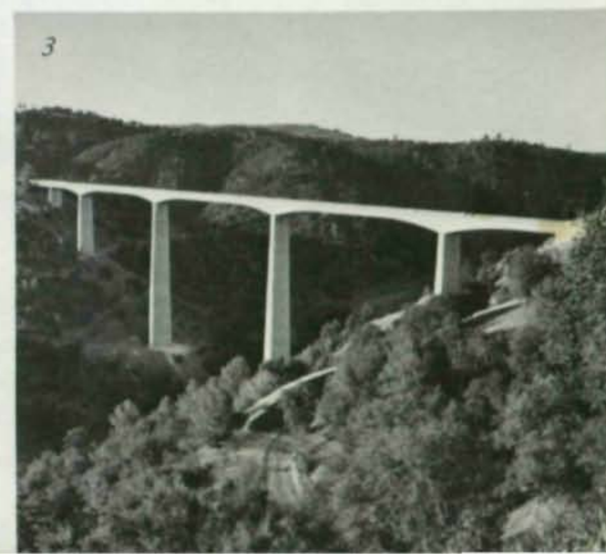

MODERN STEEL CONSTRUCTION

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CONTENTS

8th EDITION, MANUAL OF STEEL CONSTRUCTION

Copies of the 8th Edition of the AISC **Manual of Steel Construction** *are expected to be available in June 1980.*

The new edition is being extensively revised to keep pace with the many new developments in steel construction since the 7th Edition was published in 1970.

An announcement of the new 8th Edition appears on the outside back cover of this issue of **Modern Steel Construction.**

We suggest you use the convenient clip coupon to order your Manual **now** *to assure early delivery when publication is completed.*

32nd ANNUAL AISC NATIONAL ENGINEERING CONFERENCE

Leading authorities in the fields of steel design, research, and construction will meet in Pittsburgh, Pa. at the William Penn Hotel on April 29 and 30, and May 1, to exchange ideas and information. Although the program will not be announced until a later date, we can assure you that the engineer or architect who wishes to keep informed about the continuing developments in these fields will find this conference a valuable, informative, and stimulating experience.

1980 PRIZE BRIDGE COMPETITION

Entries are invited for AISC's 50th Prize Bridge Competition to select the most beautiful steel bridges opened to traffic during the calendar years 1978 and 1979. Entries must be postmarked prior to May 25, 1980. Selection of the winners will be made by a distinguished panel of professionals who will judge the entries on June 10. The members of the Jury of Awards will be announced at a later date. Further details of the competition and entry forms can be obtained from the AISC, Awards Committee, 400 North Michigan Avenue, Chicago, IL 60611.

OUR APOLOGIES

On the back cover of the 1st/2nd Q., 1979 issue, the credits for the Bell Tower at Bryant College, Smithfield, R.I., were inadvertently omitted. They are as follows:

Architect: *The Providence Partnership, Providence, R.I.*
Structural Engineer: *Robert C. Lawrence, East Providence, R.I.*
General Contractor: *E. Turgeon Construction Co., Inc., Cranston, R.I.*
Steel Fabricator: *Providence Steel, Inc., Providence, R.I.*

A BIGGER & BETTER HARTFORD COLISEUM

by A. G. Ericksen, Jeffrey W. Coleman, and Donald T. Eyberg, Jr.

In the early morning hours of January 18, 1978, the roof of the Hartford Civic Center Coliseum collapsed. Only hours before, 4,000 people had attended a basketball game in the coliseum. The collapse left the general public in shock, the then New England Whalers Hockey Team without a home, and the City of Hartford without its main drawing card in a complex consisting of a hotel, restaurants and a shopping center. Only hours after the collapse, City officials declared that the coliseum would be rebuilt "Bigger and Better." This was to be the theme and driving force that would result in not just a new roof for an old coliseum, but an expanded and vastly improved facility tailor-made to the needs of the Hartford community and Civic Center staff.

After quickly selecting Lev Zetlin Associates, Buck and Buck, and Loomis and Loomis to investigate the collapse, the City turned their attention to the selection of an Architect and Construction Manager for the reconstruction effort. On February 23, 1978, 36 days after the collapse, the City chose Ellerbe Associates, of Bloomington, Minnesota, as Architect and Engineers, and the George A. Fuller Company, of New York City, as Construction Manager. The selections were made through a competitive process.

On February 27, 1978, Ellerbe Associates and the Fuller Company opened a joint office in Hartford for the purpose of developing alternate redesigns and accompanying schedules and budgets for comparison by the City. Concurrently with this effort, the City began soliciting reconstruction funds from federal and state agencies, as well as negotiating with the coliseum's insurance carrier, The Travelers Insurance Company. It is interesting to note that, to date, no direct local tax dollars in the form of bond issues have been required for the reconstruction and enlargement of the coliseum.

Design Considerations

During the initial design phase, Ellerbe had five architects and eight engineers, working shoulder to shoulder with three estimators and schedulers from the Fuller Company, developing a design that would fit the anticipated budget and a scheduled completion date of late 1979.

Even disregarding the requirement for increased seating capacity and increased building size, the replacement of the coliseum roof posed some interesting complexities. The original roof structure was a space frame 12' deep and 360' long by 300' wide. It was supported by four pylons measuring 7' x 7' at the top, and 8' x 8' at the base. The pylons are socketed into the bedrock below. The spacing of the pylons was 210' x 270', leaving a 45' roof cantilever in two directions at each pylon. It was decided from the start that, in order to renew confidence in the structural integrity of the

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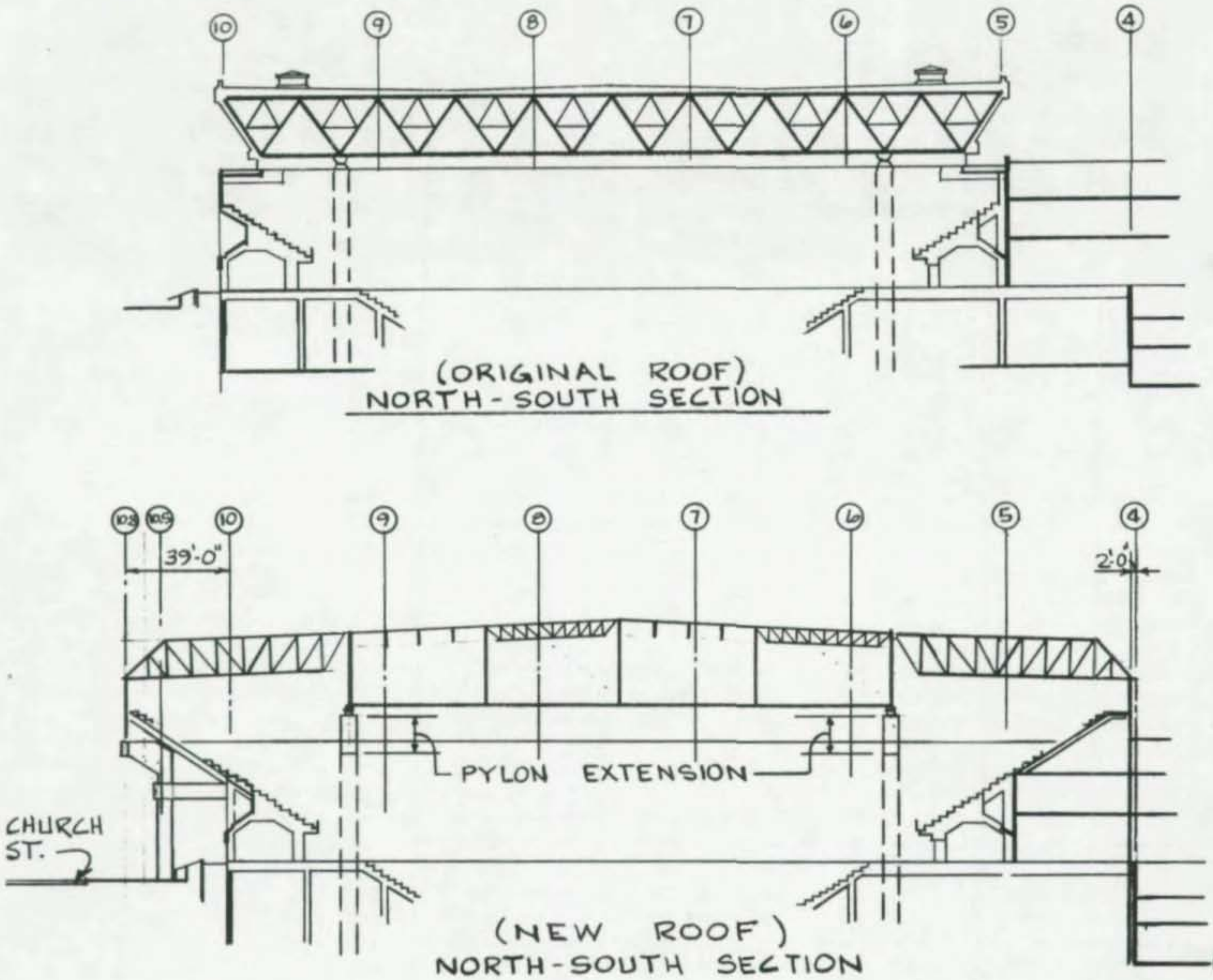
Donald T. Eyberg, Jr. is Architect, Ellerbe Associates, and was Project Manager on the Hartford Coliseum reconstruction.

new colisuem, a conventional structural system should be incorporated into the reconstruction plan.

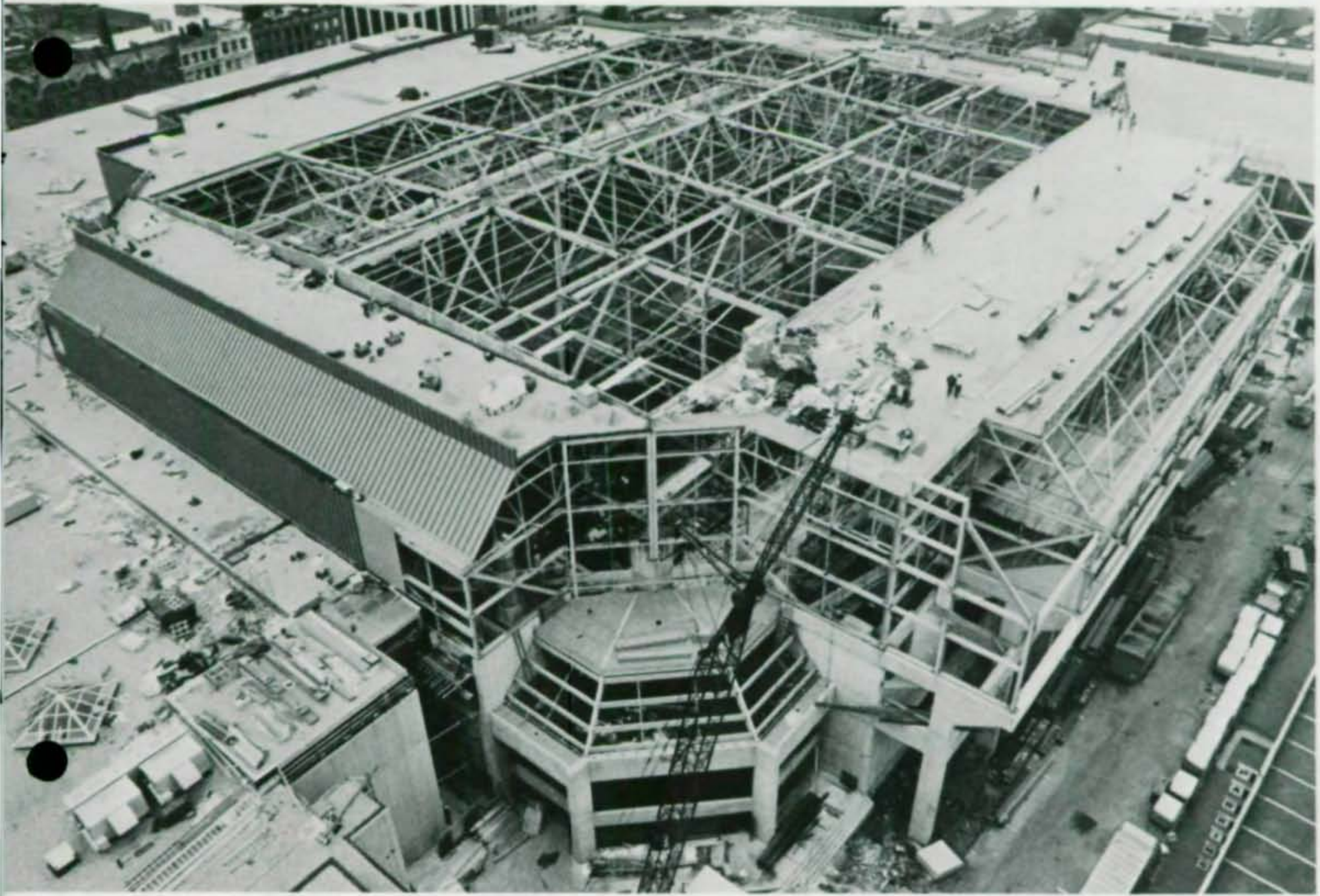
The reuse of the four original pylons was a key issue in determining the new structural system. Therefore, the four main pylons underwent an extensive testing program to determine damage, if any, caused by the collapse, as well as to verify concrete strength and rebar size and location. This was accomplished through the use of concrete core testing, Windsor probe tests and a visual inspection for cracks. Reinforcing was mapped, using a magnetic reinforcing bar locator. The pylons were found to be undamaged by the collapse and well above original design strength requirements.

Even with the reuse of the pylons apparent, the first choice for roof replacement was a system of one-way trusses spanning north-south. However, due to the length of span required, it was determined that this system wasn't the most cost effective. After reviewing the many possibilities, Ellerbe's engineers and designers decided on a rectangular two-way truss system, 210' x 270', with each of the four corners setting on one of the original pylons. The perimeter areas, part of which were originally covered by the 45' space frame cantilever, are now covered by one-way simple Pratt trusses spanning from the two-way truss system outward to new perimeter columns. In this manner, the original seating extends out into new column-free space on three sides of the building.

The coliseum is bordered on two sides by the balance of the Civic Center complex, consisting of retail shops, restaurants and, on the third floor level, offices of the Aetna Insurance Company. In order to expand the seating capacity from the original 10,500 fixed seats to the proposed 15,000 (final capacity 14,600) seats, it was necessary to encroach on the Aetna Insurance Company space at the



Comparison of original and new roof construction.



Reconstruction underway on the new Coliseum.

south border of the coliseum. An agreement was reached between the City of Hartford and Aetna officials whereby a 50' x 108' strip of office space became part of the colisuem. The south seating section extends past the original building line into the new areas where it is supported, along with the south roof area, by columns that pass vertically nearly 100' through retail shops, offices, and exhibit hall space to new foundations below. The placement of the columns was accomplished in a unique manner. First, enclosures were built at each level in the retail shop or offices, where a column was to pass. The enclosures allowed much of the work that followed to be accomplished during normal working hours with no interruption in business routines. Next, holes in the floor slabs were cut, creating a vertical "tunnel." Finally, the steel column sections were lowered from the roof level to the newly constructed foundations and later spliced by welding.

On the west and north sides of the coliseum, the seating was also extended out beyond the original building line, up the curb line and over the street. New support for these seating sections was provided by concrete frames on caissons on the west elevation, and conventional foundations on the north elevation. On the north side, the lowest level of the coliseum was extended out to the new column line, providing additional space for use by the owner.

In addition to increasing the seating capacity, the interior of the coliseum has received a complete overhaul that includes the following: the addition of handrails in the aisles, an increase in restroom and concession facilities, the inclusion of wheelchair ramps and increased wheelchair seating, a new fire detection system, including smoke evacuation capabilities, and other major improvements.

Structural System

The two-way truss system consists of five 270' Warren trusses spanning in the east-west direction, and six 210' Pratt trusses in the north-south direction. The system was completely constructed on shoring towers and 100% bolted prior to release of the towers, ensuring the desired two-way action. The truss types (Warren E-W vs. Pratt N-S) were varied to provide a more uniform two-way distribution of forces, as well as to avoid a difficult detail problem by having only two diagonals frame into each connection point. The five east-west trusses are 52'-6" apart, and the six north-south trusses are 54'-0" apart, forming 20 bays 52'-6" x 54'-0". Deep longspan bar joists are used to fill in the bays, with joists spanning alternate directions in adjacent bays, again to ensure true two-way distribution. Three-in. steel decking spans between the steel joists.

The two-way truss system is made up of 360 wide-flange sections varying from W14x61 to W14x550. All members are oriented with their webs horizontal, allowing the flanges to be connected with vertical gusset plates. Total weight of the two-way system is approximately 1,200 tons. All bolts used in the system are ASTM A490; all other bolts outside of the two-way system are ASTM A325. The live load capacity exceeds the local building code requirement of 30 lbs/sq ft by 5 lbs/sq ft. The roof will withstand a total applied load of 5,660,000 lbs. Dimensions of the total roof, including perimeter areas, are 358' x 420'.

Construction of the roof and other steel erection was performed by Karl Koch Erecting Company, Inc., of Carteret, New Jersey, the company that played a maior part in the construction of the World Trade Center in New York City. For erection of the roof, they chose a Manitowac 4100 Tower crane with a 160' mast, 150' boom and an additional 30' jib. The crane was assembled on the arena floor, since there was no entrance available for such a machine. The perimeter trusses of the two-way system were erected first in conjunction with their corresponding perimeter roof which also served as bracing for each truss. After completion of the perimeter roofs, the two-way system was erected leaving one bay, 52'-6" x 210'-0", open to allow lowering of the crane boom. This last bay was erected by a smaller hydraulic crane. Steel decking, roofing and metal wall paneling followed closely behind each phase of the steel erection.

Design of the two-way system was accomplished through the use of two computer programs, "Stress" and "Strudle", as well as manual computations. Truss connections and details were designed and detailed in a joint effort by the engineers for Ellerbe and the detailers in the fabrication office, which allowed Koch to tailor-make the connections to meet the capabilities of their fabrication shop, as well as speed up the shop drawing review process.

The City of Hartford, in an effort to verify and ensure the integrity of the design, hired a local structural design firm to perform an independent structural analysis. This firm, Burton & VanHouten Engineers, of West Hartford, Connecticut, also used the Strudle program in the process of their review. However, all loads and other variable information were arrived at independently, with only the end results checked for comparison. In addition, Burton & VanHouten checked all other new structural design work, including concrete frames and foundations.

Inspection

During construction, Minges Materials Testing Laboratories, Inc. of Avon, Connecticut, handled all phases of inspection with the exception of structural steel, which was subcontracted by Minges to Non-Destructive Test Engineering Division of Hartford Steam Boiler & Inspection Company, Essex, Connecticut. Inspection was also a duty of the Construction Manager. In addition, the construction was observed by members of the City's Department of Licensing and Inspection, as well as by two full-time structural engineers; one provided by Burton & VanHouten and one provided by Ellerbe Associates.

Structural steel inspection was performed both in the fabrication shop and in the field. These inspections went considerably beyond the normal building construction inspection procedures. All forms of non-destructive testing were employed at one time or another, including ultrasonic, magnetic particle, X-ray, dye-penetrant, visual inspection of welds, and torque testing and installation inspection of high strength bolts. Non-Destructive Test Engineering had a field staff of five inspectors who worked daily with the ironworkers to identify and correct problem areas as they arose.

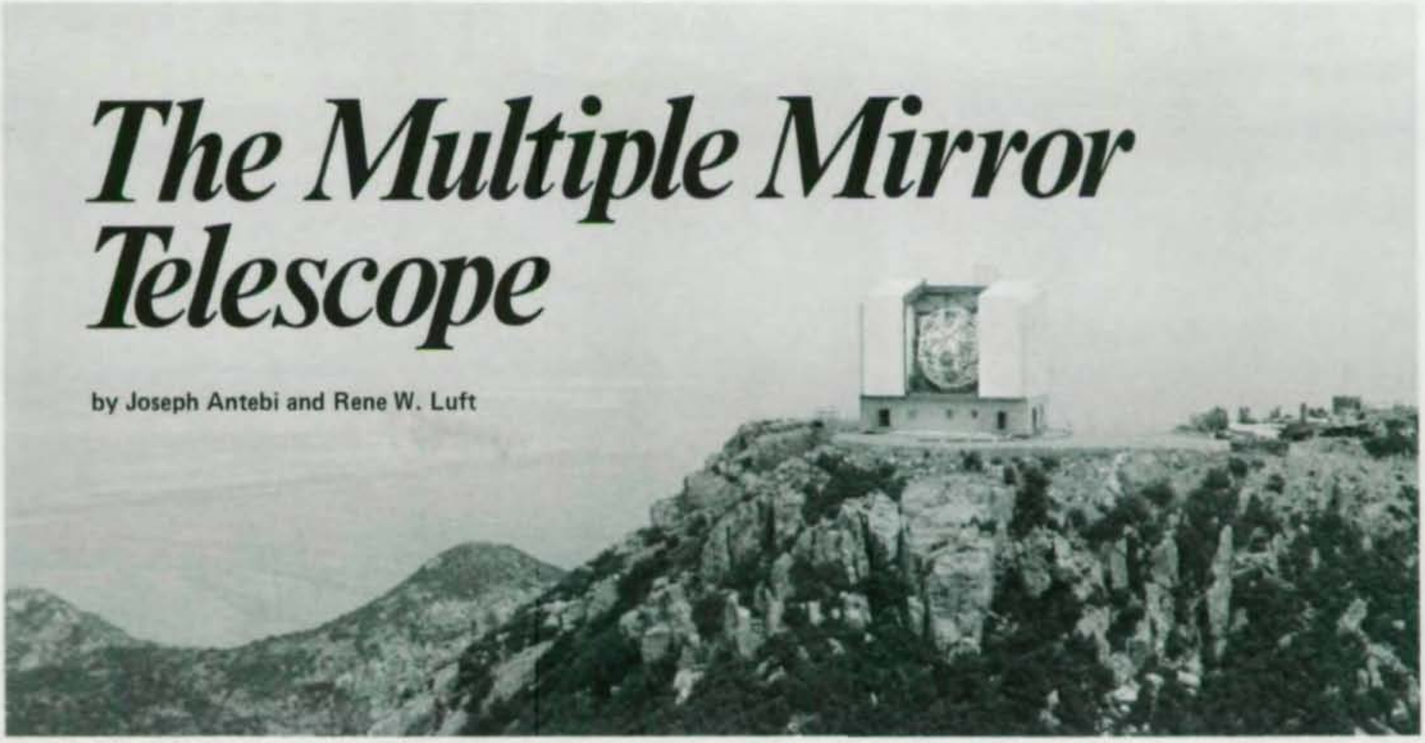
Completion

On October 15, 1979, 21 months after the collapse of the original coliseum roof, the new roof was swung free from its shoring towers. The dead load truss deflections, as verified by the engineers, indicated the roof to be performing as designed. The construction effort is now directed toward completing the mechanical, electrical and interior finishes, which couldn't be worked on until the building was weather-tight. The coliseum opening is presently scheduled for January 17, 1980; with a new roof and a vastly enlarged and improved facility, the people of the City of Hartford are sure to enjoy their "Bigger and Better" Coliseum.

View of the Coliseum before reconstruction began.

The Multiple Mirror Telescope

by Joseph Antebi and Rene W. Luft



Perched atop Mt. Hopkins, the entire telescope complex rotates on a flat hardened-steel track.

The Multiple Mirror Telescope on Mt. Hopkins in Arizona, dedicated on May 9, 1979, is the world's third largest optical telescope, with an effective aperture of 176 in. This telescope, which is of a new and unconventional design, is a joint project of the Smithsonian Astrophysical Observatory and the University of Arizona. It was built for under $8 million, which is estimated to be one-third to one-half the cost of an equivalent facility of conventional design. Thus, if it is as successful as initial experiments indicate, the Multiple Mirror Telescope (MMT) may be the first of a new generation of much larger telescopes.

To date all large telescopes utilize large parabolic reflectors ("primary mirrors") to focus the light. The astronomer's need for ever more powerful optical telescopes has until now been limited by the size of the largest primary mirror that could be fabricated and transported. The MMT circumvents this limitation by combining at a common focus the images from six telescopes of conventional optical layout, mounted in parallel on a common support. A servo-control system is used to accurately superimpose the images; but, since the range of the servo-system is limited, the support structure must maintain the relative displacements of the optical elements within a few thousandths of an inch.

The other major unconventional features of the MMT are the use of an altitude-over-azimuth mount instead of the traditional equatorial mount, and a rotating building to house the telescope; the resulting facility is functional and economical.

This paper is concerned with the structural aspects of the MMT, primarily the optical support structure, the mount, and the rotating building.

General Configuration

The major optical components are six 72" diameter primary mirrors symmetrically arranged in a hexagonal pattern around a guide/alignment telescope. A 10" diameter secondary mirror is located 14' in front of each primary mirror. The starlight from each of the six telescopes is reflected, by small flat mirrors mounted at the center of each primary, to a beam combiner and thence to a common focus on the central axis of the hexagon.

To maintain the telescopes aligned and their images superimposed, each of the 10" diameter secondary mirrors can be tilted about two axes and translated axially under servo-control. The detection system for the servo uses laser beams; these start at the guide/alignment telescope, and fall onto detectors at the beam combiner after tracing paths parallel to the starlight through the six telescopes. The design requirement is to maintain the images superimposed to within one arc second; the active optics can do this only if the uncorrected displacements of the system are very small and if the structure has no natural frequencies that will couple with those of the servo-system.

The optical elements are supported by the optical support structure (OSS), which itself is supported by an altitude-over-azimuth mount structure. The mount, somewhat as in a theodolite, rotates about a vertical, or azimuth, axis and allows the OSS to rotate in altitude about a horizontal axis.

Optical telescopes have traditionally used equatorial mounts so that the telescope could track a star smoothly with a constant velocity drive about one axis. In contrast, an altitude-over-azimuth, or alt-azimuth, mount is simpler to design and construct, because gravity forces do not vary with azimuth rotations; however, the tracking of a star requires simultaneous variable speed rotation about both axes. This is now readily achievable by using computer controlled drives, which have been used successfully for radio telescopes and radar antennas.

The decision to use an alt-azimuth configuration also affected the building choice. In a conventional observatory, the telescope is at the top of the building, under a rotating dome, and the floor of the telescope room must be kept clear so as not to obstruct the telescope or its line

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Joseph Antebi and Rene W. Luft are Principal and Associate of Simpson Gumpertz & Heger Inc., Consulting Engineers, Cambridge Massachusetts.

of sight. At the MMT, the building surrounds the telescope on three sides and rotates with it, so that the rotating building is considerably smaller than an equivalent conventional building and costs correspondingly less. An additional benefit of this configuration is that laboratories and the control and observing rooms remain adjacent to the telescope as it rotates.

The Optical Support Structure

The OSS approximates in outline a 20' cube whose function is to support within it the optical elements.

The primary mirrors are of a new lightweight honeycomb construction; each mirror together with its housing weighs about 4,000 lbs. Solid mirrors would have weighed considerably more, with a consequent multiplying effect on the weight of the entire telescope.

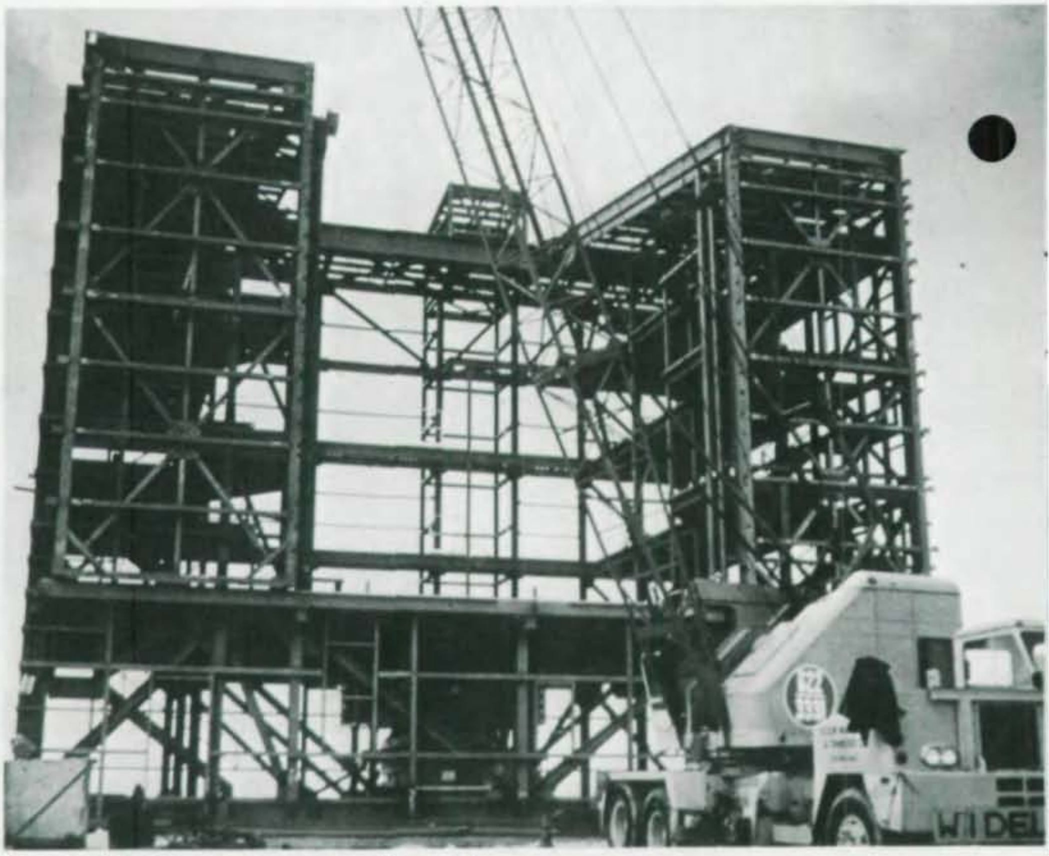
Conceptually, the OSS spans about 20' between the mount arms and supports a "useful" load—the optical elements—of about 28,000 lbs. The requirements on natural frequency limit the maximum allowable static deflection due to gravity loads to about 0.02"; this is about ten times the allowable relative displacements between a primary mirror and a corresponding secondary mirror. Two of the design objectives, therefore, were to develop a structure that would have the required absolute stiffness and that would permit a tight control of relative displacements of the mirror support points.

Other major considerations in the designs were:

- The large unobstructed apertures in front of the mirrors and through the OSS to the combined focus.
- The tight clearances within the OSS and between it and the mount.
- The need to minimize the thermal lag of all structural members, to reduce thermal distortions caused by temperature transients.
- The desire to use standard structural shapes to reduce fabrication costs.

For maximum stiffness-to-weight ratio a space truss was selected. Steel and aluminum were considered; although they have the same stiffness-to-weight ratio, steel was selected because it provides more stiffness per unit cost, and it has a coefficient of thermal expansion about one-half that of aluminum.

The structure uses standard rolled structural steel members and consists primarily of mutually orthogonal intersecting plane trusses. In its final configuration, the structure of the OSS weighs about 70,000 lbs, or 2.5 times the weight of the optical elements.



The MMT building is a 4-story steel braced frame structure.

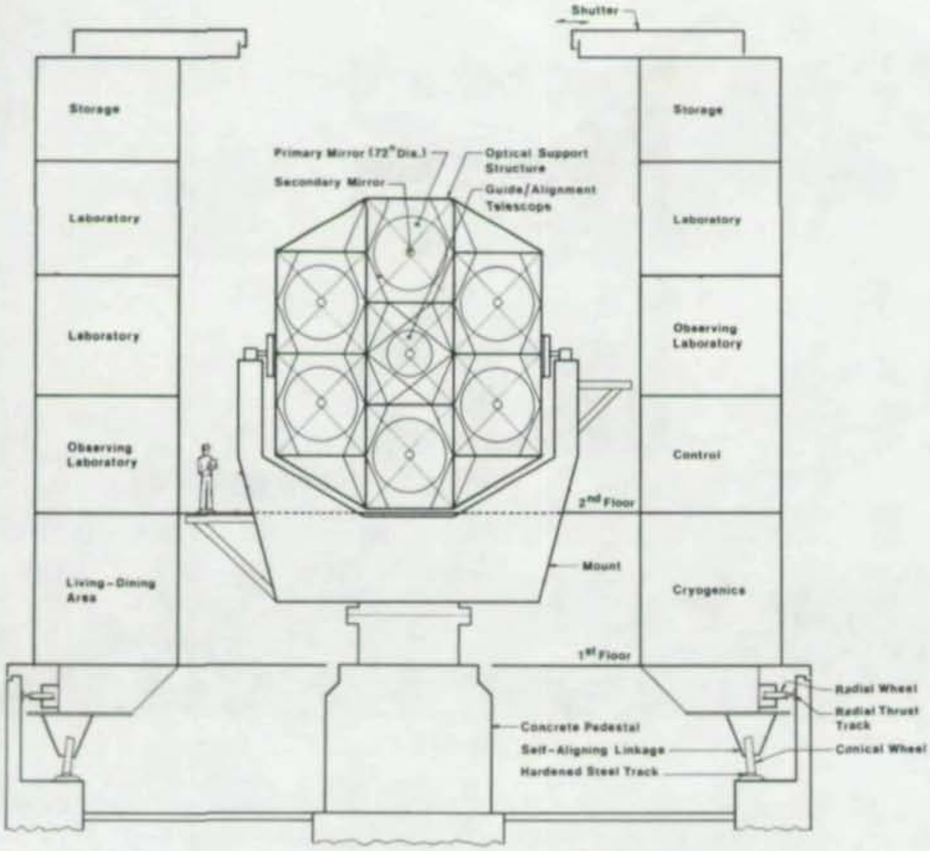
To control the relative displacements of the optical elements, independent load paths are provided for the gravity loads corresponding to the zenith- and horizon-pointing orientations of the telescope. The stiffnesses of these load paths were adjusted during the design-analysis cycle to minimize the relative displacements of the mirrors.

Since axial load paths are provided throughout the structure, the effects of bending stiffness are small and the uncertainties concerning bending stiffness of joints become negligible. This is important, since the structure must not only have small deflections, but the deflections must be repeatable and subject to accurate prediction.

Particular attention was taken in the design of joints to eliminate local flexibilities. For example, a precompressed flange connection is used for tubular members; in this design a direct tension/compression path is provided through a spacer which is compressed when the bolts that connect the flanges are torqued. Under a tensile load the precompression in the spacer is reduced; the flexibility of the flanges does not enter into the load path and the axial stiffness of the member is unaffected by the joint.

Member eccentricities at the joints can substantially reduce the effective stiffness of the structure; however, analyses showed that the reduction in stiffness can be neglected if the allowable eccentricities due to fabrication tolerances are less than 1/8"; this was achieved using conventional fabrication techniques, with good workmanship and tight quality control.

In addition to the deviations from nominal dimensions due to fabrication tolerances, the deflections may differ from the values predicted in the analysis due to variations in the member cross-sectional areas from their nominal values, and due to approximations made in the analysis. To compensate for these effects, certain members with adjustable stiffnesses were incorporated into the design so that the OSS can be "tuned" after final assembly. The members with adjustable stiffness have bolted-on cover plates which can be changed for thicker or thinner plates. The stiffness can be adjusted in smaller increments by varying the effective length of a cover plate; this is achieved by using predrilled bolt holes and varying the distance between the end of a cover plate and the first bolt.



Schematic of Multiple Mirror Telescope and rotating building.

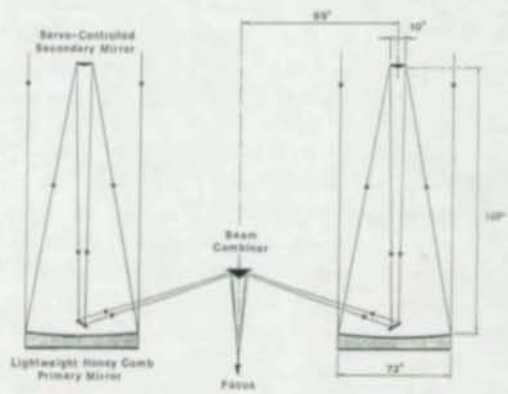
Subassemblies of the Optical Support Structure (OSS).

Initial use of the telescope for astronomical observations indicates that the relative distortions of the OSS are well within design specifications. Thus the capability of tuning the structure, although available, may never have to be used.

The Mount

The mount (see schematic illustration) has two vertical arms, at the top of which are the altitude bearings which support the optical support structure. The bases of the vertical arms are connected by a horizontal crossarm, which in turn is supported on a vertical cylinder. This cylinder rotates on an azimuth bearing mounted on a short steel cylinder supported by a concrete pier built on bedrock. The mount is a heavy, stiffened steel box weldment weighing approximately 140 tons.

The unusual feature of the mount design is the selection of a rolling-element mechanical bearing for the azimuth axis. Initially, it appeared that the required low friction and smoothness could be obtained only with a hydrostatic bearing;



Schematic of optical layout - (section is through two of six telescopes).

however, it was found that a ball thrust-bearing would meet the requirements if particularly tight tolerances (3×10^{-4} in.) could be achieved on the waviness of the raceway and the out-of-flatness of the mounting surface. Although difficult, this was successfully accomplished and resulted in a major cost saving by eliminating the need for a hydrostatic bearing.

The Building

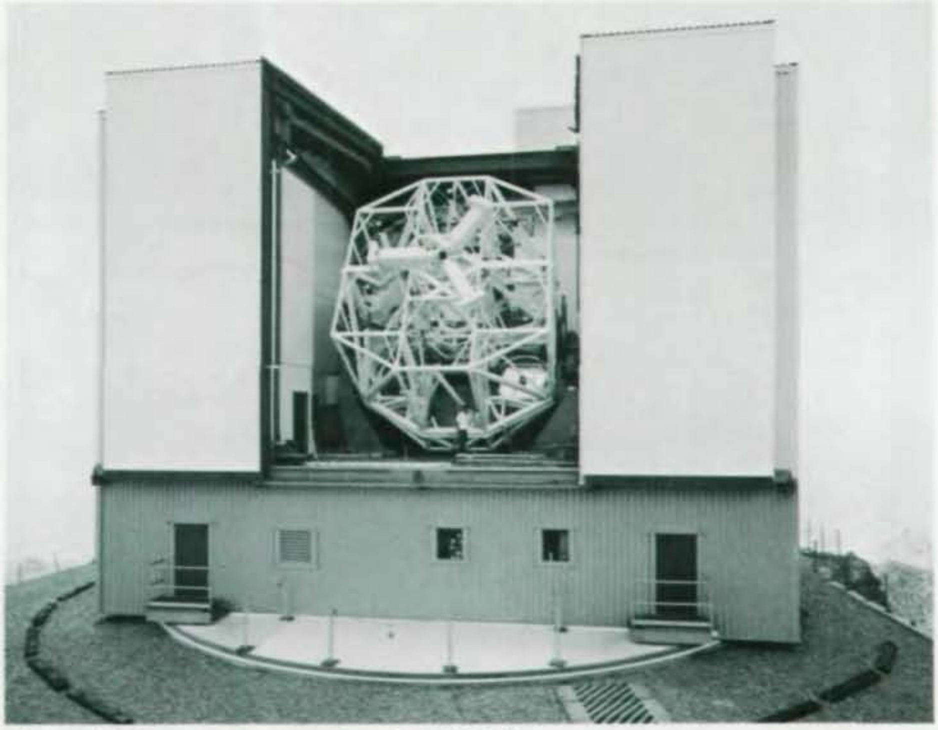
The primary function of a telescope building is to provide a protective enclosure which can be opened to expose the telescope to the sky for astronomical observations. The building must withstand the high winds associated with a storm on a mountain top, and the telescope must be usable on clear windy nights. For the MMT, at 8500′ elevation on Mt. Hopkins, the maximum design wind for survival was set at 140 mph, and 45 mph was set as the maximum wind for operational conditions. The loads associated with these winds, together with the need to have a large viewing aperture with a retractable cover, imposed severe constraints on the design of the building.

The selection of an alt-azimuth mount for the MMT led to the novel concept of using a rotating building. With such a building, the telescope room need only be large enough to allow the OSS to rotate in altitude, and provide a clearance for small relative motions in azimuth. The width of the required viewing aperture for the MMT is slightly wider than the OSS; the entire roof and the front wall of the telescope room must be retractable to allow viewing from zenith to horizon.

The space for laboratories, control rooms, and other required functions of the observatory is provided in two rectangular blocks on either side of the telescope room. Also available, as in a conventional observatory, is the space below the telescope room.

The building is essentially a compact rectangular four-story structure, 64′ x 44′ in plan and 55′ high. The viewing aperture, which is in the longer side of the building, is 30′ wide and extends from the second floor, up to the roof, and across the roof to the ridge line, 8′ from the rear wall. Bi-parting shutters cover the aperture; they are supported at the second floor level and at the ridge line on rollers which allow the shutters to move laterally. In the open position the shutters do not extend past the outside edges of the building; such protrusions would have caused additional wind loads.

The building, which weights about 500 tons, is supported on four wheels which run on a 57′ diameter flat hardened-steel track; the wheels are at the corners of a 40′ square. To roll without slipping on the flat circular track, the 36″ diameter, 4″ wide steel wheels, are conical. A proprietary self-aligning linkage adjusts the wheel position for track irregularities to minimize the contact stresses at the rail surface; this allows the use of wheel loads of the order of 170 tons, which correspond to the design condition for winds of 140 mph. Lateral loads are carried by four horizontal wheels running on a circular track on the interior wall of the circular foundation. Shear pins are provided to stow the building and prevent rotation under high winds.



The steel OSS supports approximately 28,000 lbs.

The building is a steel braced frame structure with concrete floors. The main framing consists of the four exterior walls, the roof, and the first floor, that is the six faces of the rectangular block, all acting together as interconnected plane braced frames; because of the viewing aperture, the braced frames of the roof and the front wall are U-shaped. The telescope floor at the second level provides an interior diaphragm across the entire building; above this level the floors, which extend around the three sides of the telescope room, also act as diaphragms. The first floor framing includes four 48″ deep girders forming a 40-ft square at the corners of which are the wheel assemblies on which the building is supported. The two girders which span across the width of the building lie in the planes of the side walls of the telescope room and provide support for these walls.

A feature of the telescope room floor is that it has openings for the two arms of the mount, and spans over the mount crossarm to provide a floor which is independent of the telescope itself. Since a floor depth of only 4″ was available to span 13′ over the crossarm, an orthotropic steel deck was designed.

The building is well insulated; in particular, the shutters and the walls of the telescope room are insulated with steel-skinned, foam core, sandwich panels.

A special consideration in the design of telescope buildings is their effect on the seeing conditions. These are degraded if convection currents of warm air pass across the mirror apertures; thus, the need to insulate the telescope room from the adjacent heated spaces. Provisions have also been made to cool the floor of the telescope room by circulating a refrigerant through pipes embedded in the concrete slab. The purpose of cooling the floor is to prevent the thermals which rise from a warm floor when the telescope room is opened to cooler night air.

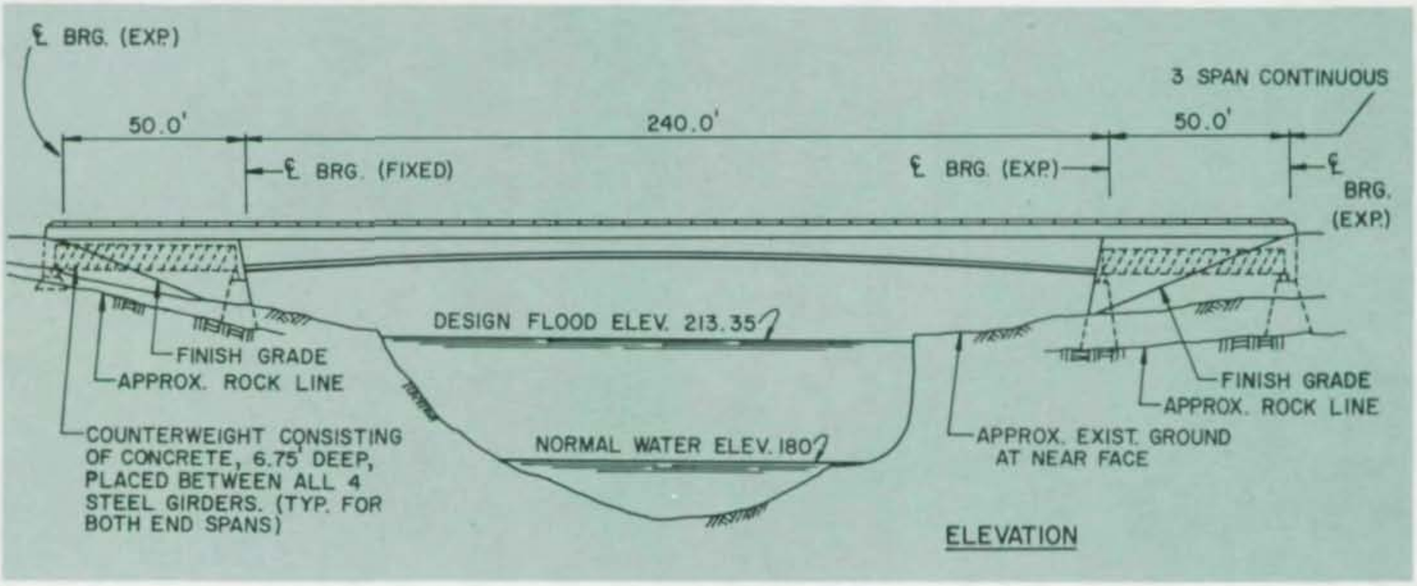
Smithsonian Astrophysical Observatory
Program Management for the Optical Support Structure, the Mount, and the Building

Simpson Gumpertz & Heger Inc.
Cambridge, Mass.
Conceptual and Preliminary Design of the Optical Support Structure; Structural Design of the Building

Wallace, Floyd, Ellenzweig, Moore Inc.
Cambridge, Mass.
Architectural Design of the Building

Western Development Laboratories
Ford Aerospace Corp.
Palo Alto, Cal.
Prime Contractors for the Optical Support Structure, The Mount, and the Building

New Bridge Enhances Historic Site



The new Lover's Leap Bridge that spans the Housatonic River in New Milford, Connecticut, is not only surrounded by great scenic beauty, but by a romantic legend as well.

Completed in October, 1977, the structure was designed to replace an old but picturesque iron truss bridge built in 1888. The old bridge, no longer safe for present day traffic, is located near a great gorge, the top of which is known as Lover's Leap. There, as legend has it, an Indian Princess, Lillinanah, unable to marry her white lover, leaped to her death. Later, her lover followed her in his own death plunge. The old bridge near the famous gorge is closed to vehicular traffic, but is listed as a Connecticut historic landmark, open to sightseers and history buffs alike.

In planning the new structure, several alternatives were considered, including a simple span plate girder, a continuous plate girder, and a truss bridge. Structural, economic, and aesthetic studies determined that a welded, composite three-span continuous counterweighted plate girder bridge offered the simplicity and slenderness necessary, as well as a savings of about $40,000 over the other schemes.

The rugged natural beauty of the Housatonic River Valley required a structure that would have the least visual impact on the waterway and surrounding area. Rock conditions in the riverbanks made a clear main span of at least 240 ft necessary, while the overall bridge length was limited to 340 ft by the geometry of approaching roads. Spaced on 10-ft centers, four plate girders support the 37.8-ft-wide roadway. Span lengths are 50, 240, and 50 ft.

The unusually short end spans, 50 ft each, were counterweighted with concrete to offset the uplift at the abutments and decrease positive moment at the center of the bridge. The span proportions with the counterweights allowed the use of a shallow depth at the center of the main span, resulting in aesthetically pleasing lines.

Each 340-ft-long continuous girder has two field splices. After erection of the 110-ft end sections, concrete counterweighting was poured, and finally, lifted by cranes, the 120-ft center section of each girder was swung into place and attached by bolted splices. Web depth for the composite girders varies from 9 ft in the end spans to 5 ft at the center of the main span.

The girders were fabricated from ASTM A588 weathering steel. When left unpainted, weathering steel develops a protective oxide coating that changes, after it has fully weathered, from a rusty brown to a rich brown tone. The weathering material was a natural choice, since it requires little maintenance and blends in so well with the natural, unpolished beauty of the new bridge site. (The new Lover's Leap Bridge won an Award of Merit in the American Institute of Steel Construction's 1978 Prize Bridge Competition in the medium span, high clearance category. See p. 15.)

The bridge's total cost was $672,000 or approximately $54 per square ft of the structure.

Owner:
Town of New Milford, Connecticut

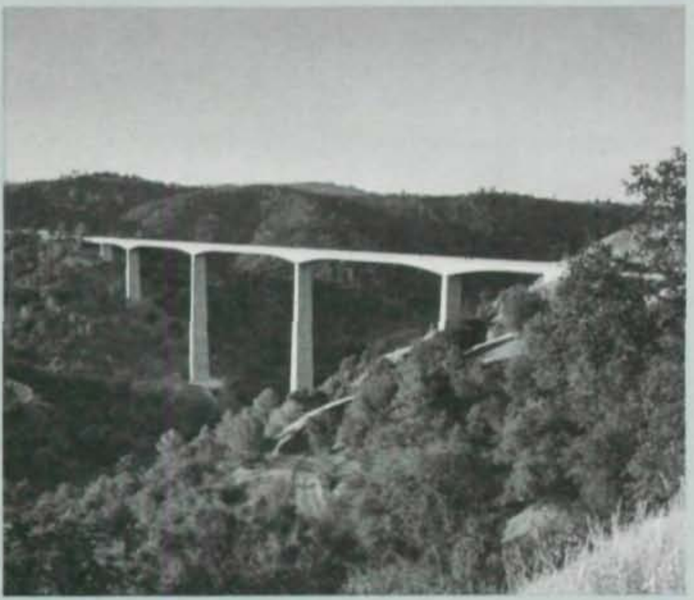
Designer:
C.D.O.T. Bridge Design Unit
Wethersfield, Connecticut

General Contractor:
The Brunalli Construction Co.
Southington, Connecticut

Fabricator:
West End Iron Works, Inc.
Cambridge, Massachusetts

1978 PRIZE BRIDGES

PRIZE BRIDGE 1978 — MEDIUM SPAN, LOW CLEARANCE
Wilder Gulch Bridge
Interstate 70, Summit County, Colorado
Designer: Meheen Corporation
Owner: Colorado Division of Highways
Architectural Consultants: Taliesin Associated Architects of Frank Lloyd Wright Foundation; Oliver and Hellgren
General Contractor: Colorado Constructors Div., Green Construction Co.
Fabricator: Burkhardt Steel Company
Erector: Colorado Constructors Div., Green Construction Co.



PRIZE BRIDGE 1978 — LONG SPAN
Archie Stevenot Bridge
State Route 49, Near Sonora, California
Designer/Owner: California Department of Transportation
General Contractor: Hensel Phelps Construction Company
Fabricator/Erector: Kaiser Steel Corporation

PRIZE BRIDGE 1978 — SHORT SPAN
Pine Road Bridge over Pennypack Creek
Philadelphia, Pennsylvania
Designer/Owner: City of Philadelphia, Department of Streets
General Contractor: Tel-Stock, Inc.
Fabricator: Williamsport Fabricators, Inc.
Erector: Cornell & Company, Inc.

PRIZE BRIDGE 1978 — MEDIUM SPAN, HIGH CLEARANCE
Polk Creek Bridges
Interstate 70, Eagle County, Colorado
Designer/Owner: Colorado Division of Highways
Architectural Consultants: Taliesin Associated Architects of Frank Lloyd Wright Foundation; Oliver and Hellgren
General Contractor: Centric Corporation
Fabricator: The Midwest Steel and Iron Works Co.
Erector: Centric Corporation

PRIZE BRIDGE 1978 — GRADE SEPARATION

Cold Springs Interchange Bridge
US 395, North of Reno, Nevada
Designer/Owner: Nevada State Highway Department
General Contractor: Robert L. Helms Construction Company
Fabricator/Erector: Utah Pacific Steel Company

PRIZE BRIDGE 1978 — ELEVATED HIGHWAYS OR VIADUCTS

Martin Luther King, Jr. Memorial Bridge
Richmond, Virginia
Designer: Parsons, Brinckerhoff, Quade & Douglas
Owner: City of Richmond
General Contractor: Central Contracting Company, Inc.
Erector: Cornell and Co., Inc.

PRIZE BRIDGE 1978 — MOVABLE SPAN

Loomis Street Drawbridge
Chicago, Illinois
Designer: Chicago Department of Public Works, Bureau of Engineering
Owner: City of Chicago
General Contractor: Paschen Contractors, Inc.
Fabricator/Erector: American Bridge Division, United States Steel

PRIZE BRIDGE 1978 — SPECIAL PURPOSE

Salina Street Bridges
Syracuse, New York
Designer: Schleicher-Soper Architects, AIA
Owner: City of Syracuse
Consultant: John P. Stopen, Structural Engineer
General Contractor: Northeast Construction Managers Corporation
Fabricator: Smith and Caffrey Steel Corp.
Erector: Onondaga Steel Erectors, Inc.

PRIZE BRIDGE 1978 — RAILROAD

L & N Railroad Bridge over Briley Parkway
Nashville, Tennessee
Designer: Clarke and Rapuano, Inc.
Owner: Tennessee Department of Transportation
General Contractor: Oman Construction Co.
Fabricator: American Bridge Division, United States Steel
Erector: Metler Crane and Erection Service

PRIZE BRIDGE 1978 — RECONSTRUCTED

Rapp's Bridge
East Pikeland Township, Pennsylvania
Designer/Owner: PennDOT
General Contractor: Bear Creek Construction Co.
Fabricator: Cumberland Bridge Company
Erector: Bear Creek Construction Co.

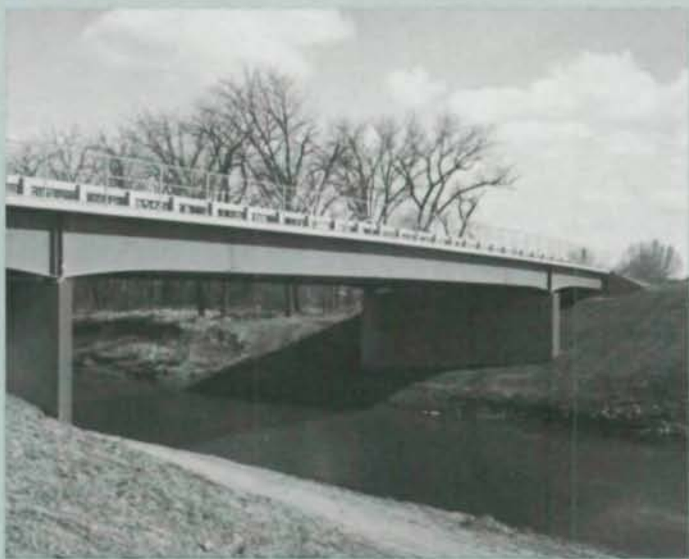
1978 PRIZE

AWARD OF MERIT 1978 — LONG SPAN
New River Gorge Bridge
Fayetteville, West Virginia
Designer: Michael Baker, Jr., Inc.
Owner: West Virginia Department of Highways
General Contractor: American Bridge Division, United States Steel
Fabricator/Erector: American Bridge Division, United States Steel

AWARD OF MERIT 1978 — LONG SPAN
Francis Scott Key Bridge
Baltimore, Maryland
Designers: Greiner Engineering Sciences, Inc.; November & Hurka; Singstad Kehart; Baltimore Transportation Associates
Owner: Maryland Transportation Authority
Engineering Consultant: Greiner Engineering Sciences, Inc.
General Contractor: Pittsburgh-Des Moines Steel Company
Fabricator: Pittsburgh-Des Moines Steel Company
Erector: John F. Beasley Construction Company

AWARD OF MERIT 1978 — MEDIUM SPAN, HIGH CLEARANCE
Lover's Leap Bridge
New Milford, Connecticut
Designer: C.D.O.T. Bridge Design Unit
Owner: Town of New Milford
General Contractor: The Brunalli Construction Co.
Fabricator: West End Iron Works, Inc.
Erector: The Brunalli Construction Co.

AWARD OF MERIT 1978 — MEDIUM SPAN, HIGH CLEARANCE
Maury River Bridges
Lexington, Virginia
Designer: Knoerle, Bender, Stone and Associates, Inc., A Division of Environdyne Engineers
Owner: Virginia Department of Highways and Transportation
General Contractor: Crowder Construction Company
Fabricator: Carolina Steel Corporation
Erector: Southern Contractors Service, Incorporated



AWARD OF MERIT 1978 — MEDIUM SPAN, LOW CLEARANCE
Sitting Bull Bridge
Mandan, North Dakota
Designer: North Dakota Stage Highway Department
Owner: State of North Dakota
General Contractor: James J. Igoe and Sons Construction Company
Fabricator: Egger Steel Company
Erector: James J. Igoe and Sons Construction Company

BRIDGES (CONT'D)

AWARD OF MERIT 1978 — SPECIAL PURPOSE
Mine Falls Park Pedestrian Bridge
Nashua, New Hampshire
Designer: Smith & Hamilton, Inc.
Owner: Nashua Park Recreation Commission
Consultant: Andrews & Clark
General Contractor: Shoals Corporation
Fabricator: Bancroft & Martin, Inc.
Erector: Shoals Corporation

AWARD OF MERIT 1978 — SHORT SPAN
I-70 Bridge over Smith Gulch
Vail Pass, Colorado
Designer: Meheen Corporation
Owner: Colorado Department of Highways
Architectural Consultant: Taliesin Associated Architects of the Frank Lloyd Wright Foundation
General Contractor: Green Construction Company
Fabricator: Burkhardt Steel Company
Erector: Green Construction Company

AWARD OF MERIT 1978 — SPECIAL PURPOSE
Manomin Park Bridge
Fridley, Minnesota
Designer: DeBourgh Manufacturing Company
Owner: County Park Systems of Anoka, Minnesota
Consultant: Dunham Associates
General Contractor: County Park Systems of Anoka, Minnesota
Fabricator: DeBourgh Manufacturing Company
Erector: County Park Systems of Anoka, Minnesota

AWARD OF MERIT 1978 — GRADE SEPARATION
Airline Highway Interchange
Baton Rouge, Louisiana
Designer: Modjeski and Masters
Owner: Louisiana Department of Transportation & Development
General Contractor: Boh Bros. Construction Co., Inc.
Fabricator: Mississippi Valley Structural Steel Co.
Erector: Sun Erection Co., Inc.

AWARD OF MERIT 1978 — RECONSTRUCTED
Bloomington Ferry Replacement Bridge
CSAH 18 at the Hennepin County-Scott County line, Minnesota
Designer: Howard Needles Tammen & Bergendoff
Owner: Hennepin County and Scott County
General Contractor: Johnson Bros. Corporation
Fabricator: Saint Paul Structural Steel Co.
Erector: Johnson Bros. Corporation

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