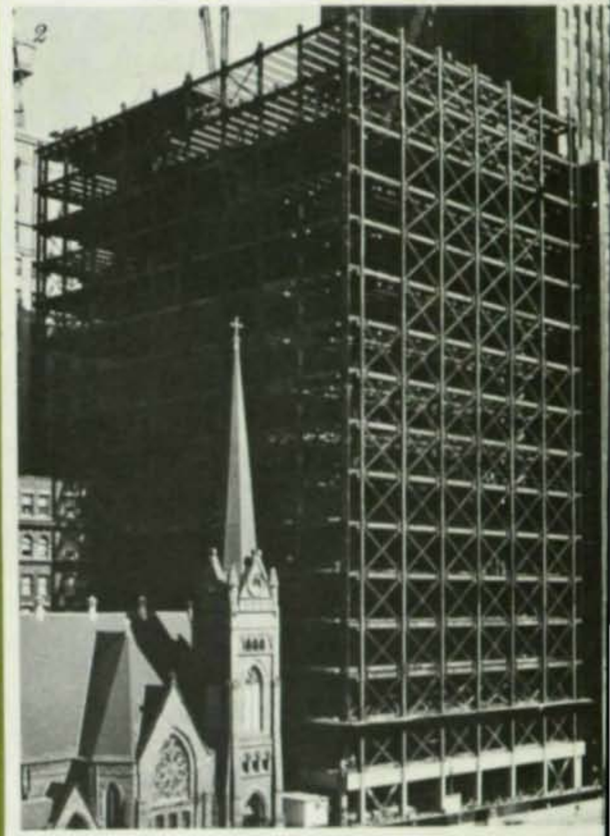
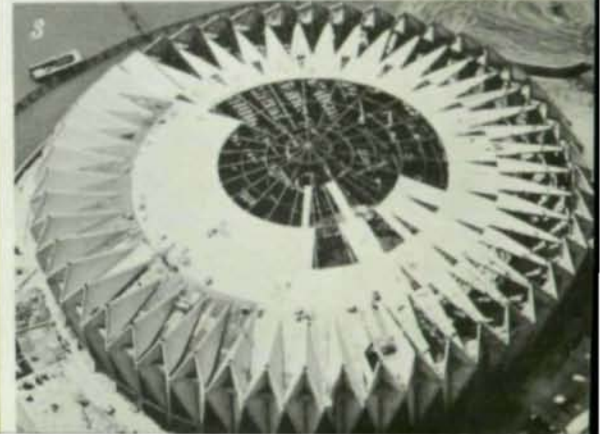
VOLUME X / NUMBER 1 / FIRST QUARTER 1970

MODERN STEEL CONSTRUCTION





MODERN STEEL CONSTRUCTION

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VOLUME X / NUMBER 1 / FIRST QUARTER 1970

CONTENTS

1970 PRIZE BRIDGE COMPETITION

Entries are invited for the 42nd Annual Prize Bridge Competition to select the most beautiful steel bridges opened to traffic during the calendar year 1969.

The members of the 1970 Prize Bridge Jury are:

Samuel S. Baxter, F.ASCE *President-elect, ASCE and Water Department Commissioner, City of Philadelphia, Pa.*

Dr. James Chinn, M.ASCE *Professor of Civil Engineering, University of Colorado, Boulder, Colo.*

Wayne S. Hertzka, FAIA *Hertzka & Knowles, San Francisco, Calif.*

Frank M. Masters, Jr. M.ASCE *Consulting Engineer, Harrisburg, Pa.*

Francis C. Turner, F.ASCE *Federal Highway Administrator, U. S. Department of Transportation, Washington, D. C.*

MANUAL OF STEEL CONSTRUCTION - 7th Edition

Copies of the 7th Edition of the AISC Manual of Steel Construction are expected to be available in July, 1970.

The new edition is being extensively revised and expanded to keep pace with the many new developments in steel construction since the 6th Edition was published in 1963.

A brochure describing the new Manual, including an order form, will be mailed to readers of MSC within the next few weeks. We suggest you order promptly to assure delivery of your copy as early as possible after publication.

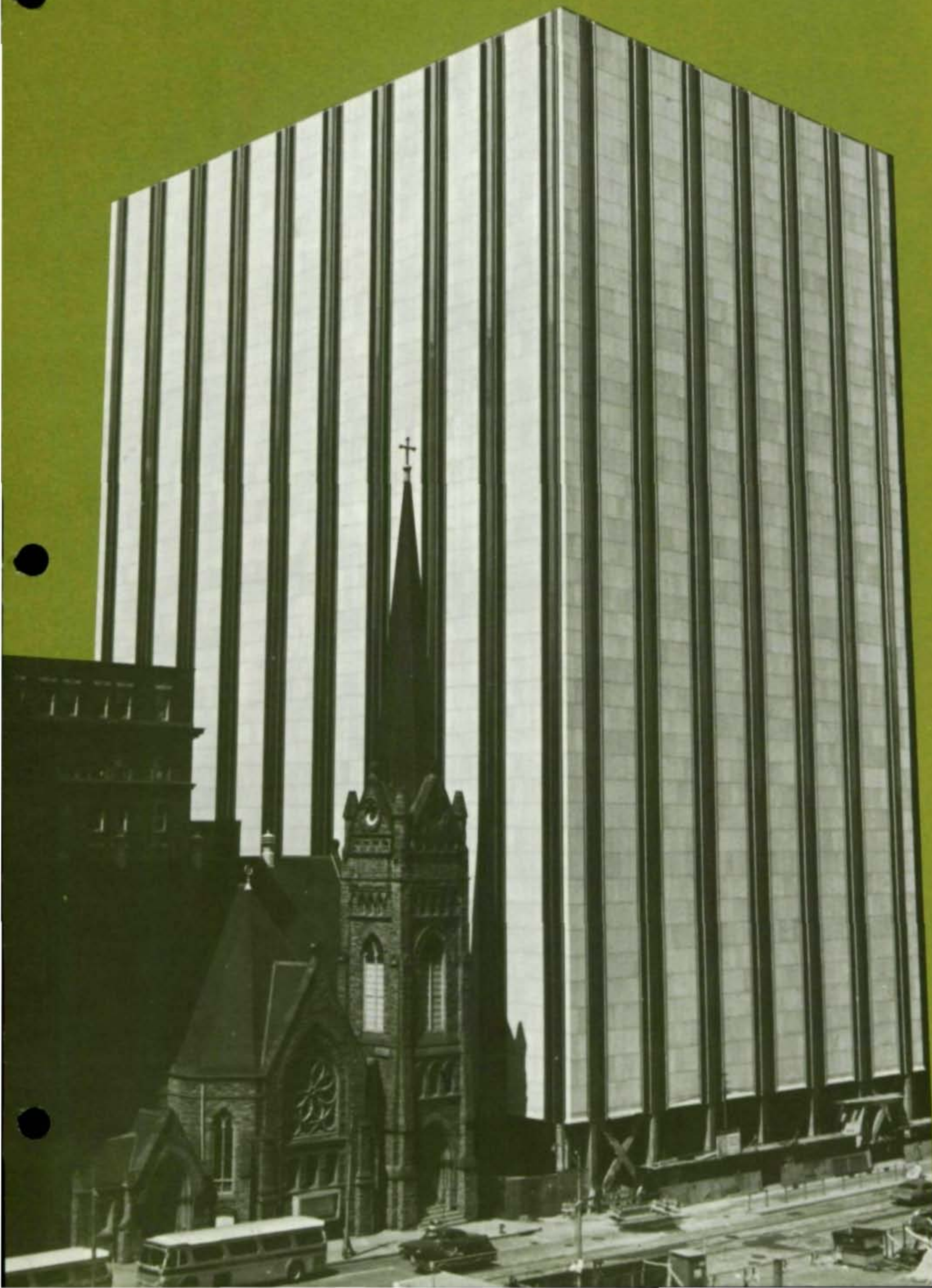
OUR APOLOGIES

On page 7 of the 4th Q., 1969 issue of MSC, the caption identifying the Jury of Awards for the AAE competition reversed the identities of Mr. John Dinkeloo and Mr. Jacques C. Brownson.

In the same issue on page 15, the first paragraph contained an equation which should have read:

$$I_{avg} = \frac{I_{ab} \times L_{ab} + I_{bc} \times L_{bc} + \cdots\cdots}{L}$$

X-Bracing System for Bell Telephone



Gap between column attachments and upper ends of X-brace members helps prevent "deformation" of members in the event framework is subjected to unusually severe horizontal loads.

There's more than talk coming out of Bell Telephone these days. A case in point is the company's new equipment facility in Pittsburgh — an atypical high-rise structure.

Consider the design requirements:

- an aesthetically pleasing 400-ft high structure, capable of withstanding unusually high floor loads.
- a disaster-proof facility in which personnel and communication can function for an extended period during an emergency.
- capable of vertical expansion to 24 stories. (The initial structure was to be 11 stories, but after construction began it went up to 16 stories, when AT&T decided to place an Overseas Operation Center in Pittsburgh.)
- Maximum usable floor area.

These design conditions presented major problems to Larsen and Ludwig, Engineers and Architects.

Unusual Loading Considerations

Floor loadings throughout the structure varied in superimposed loads (live load) from 150 lbs per sq ft to 300 lbs per sq ft. The top floor, 410 feet above street level required a live load of 300 lbs per sq ft.

Many floors required loads for equipment five times greater than the normal floor load in a high-rise office structure. Electric energy loads on some floors dissipate heat through communications equipment to the extent of 44 watts per sq ft — requiring continuous year-round air-conditioning.

Windload resistance is 150 lbs per sq ft — higher than necessary for the Pittsburgh Building Code. Floor-to-floor story heights vary from 22 ft to 17 ft-6 in. The 138-ft x 195-ft building plan called for a remarkable 82 percent productive floor space.

A conventional steel-framed structure with moment connections was discarded after several trial designs because it would require using floor girders more than 48 in. deep. Moments introduced into the columns with high story heights required that heavy plates be welded to the largest column sections rolled. This analysis actually increased story heights somewhat because of the floor-to-ceiling requirements.

With the specific requirements of this building, it was decided to investigate the use of high-strength steel as tension members to resist the horizontal forces applied to the structure. Such a design, however, could only be considered if proper connection details and welding techniques could be resolved.

After months of concentrated effort and numerous consultations on welding, strength of metals, and fabricating technique, a plan was decided upon as workable. The final solution was represented in what may be described as an X-braced structure.

Windbracing

The key to this structure is an all-important design detail in which an X-bracing system utilizes high-strength steels, specifically ASTM A514 with 100,000 psi yield strength. It also included the use of ASTM A325 and A490 high-strength bolts and ASTM A572 (50,000 psi yield strength) for the welded column attachments. Welding 100,000 psi yield strength steel to function in tension for the connections was the solution.

So efficient is the X-bracing system, used with the various high-strength steels for the framework, that two additional floors were included in the building design without a change in the overall building height requirements (in comparison to a conventional unbraced frame with moment connections).

As a result, theoretically a 22-story conventionally designed structure became a 24-story X-braced structure with a 10 percent increase in floor space, and a considerable reduction in steel tonnage and cost per sq ft of usable building area.

The high-strength materials used in the framework included "jumbo" columns weighing 730 lbs per ft, the connecting girders, and 70-ft long "jumbo" shop-spliced foundation cores (all ASTM A572). Some ASTM A36 steel was used for filler beams. Building construction used five different strengths of steel.

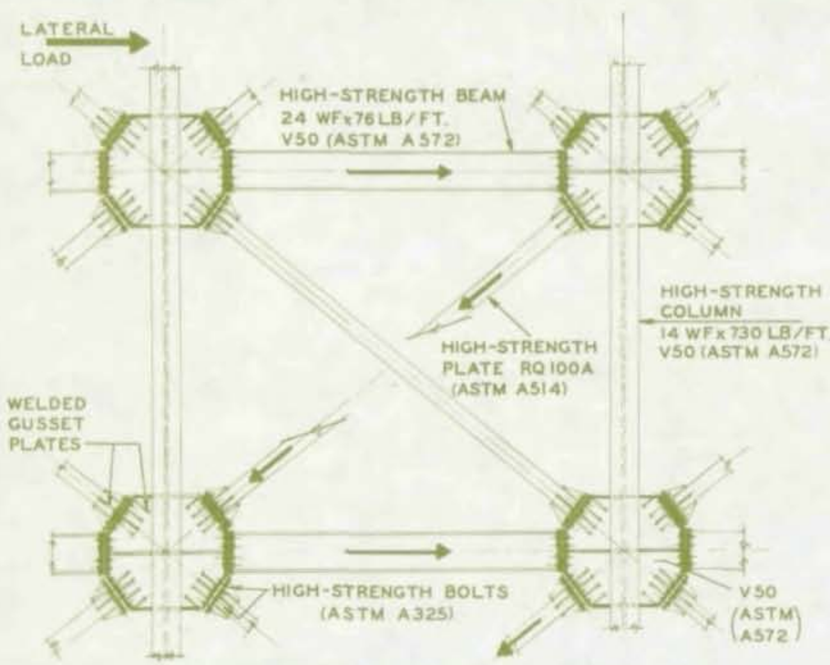
The exterior wall steel is designed to resist all horizontal loads. The interior steel carries vertical loads only. The average floor girder is an 18-in. deep WF section of ASTM A572.

Exterior walls consist of approximately 13 in. of reinforced concrete covered by 3-in. white granite slabs, adding rigidity and beauty to the structure.

ASTM A514 X-braces, 1-in. thick with variations of 6 to 16 in. in width, are selectively placed in the steel framing for maximum efficiency. Six-inch thick reinforced concrete floor slabs attached to the steel structure are designed to transmit horizontal loads through the floor framing into the X-bracing in such

Use of X-bracing members (ASTM A514 — 100,000 psi yield strength) reduced drift to a minimum.

Ironworker arc welds high-strength steel "jumbo" column (730 lbs per sq ft).



Arrows in detail indicate line of load transmission through framework when a high lateral load is applied.

Architect-Engineer: Larsen and Ludwig
Pittsburgh, Pa.
General Contractor: George A. Fuller Co., Inc.
Pittsburgh, Pa.
Steel Fabricator: Bethlehem Steel Corporation
Bethlehem, Pa.

a way that the steel framing absorbs dynamic and overturning loads. This load is then transmitted to high-strength steel columns set in solid rock 100 ft below street level.

Some of the X-bracing members are designed to carry as much as 900 tons at breaking strength.

The high-strength X-braces have welded tension fittings on the ends. The braces fit diagonally into selected exterior wall areas of the structure. They are then high-strength bolted to the column attachments where the floor spandrel beam meets the column.

The X-bracing members are shorter than the diagonal distance between the faces of the X-braced column connections. They are drawn tight to the building frame, which places the X-braced member in tension. Each 100,000 psi X-braced member is actually elongated from ⅝ to ¾ of an inch — putting a stress in that member of up to approximately ⅓ of the ultimate strength. The technique of tensioning the diagonal member to produce the initial required elongation was established on an actual full-scale building member in the fabricating shop in the early stages of the construction procedures.

As a result of ultraheavy design and pretensioning, the drift of the finished structure under normal wind loads is expected to be insignificant.

In accordance with design plans, a small amount of space is left between one end of each tensioned X-brace and the structural framework. (The space is at the top end of each X-brace.) This space, bridged by high-strength bolts, was provided for the deformation of the vertical steel due to floor loads shortening the distance between the X-bracing connections, and to retain part of the residual elongation. This space provides for the "give" necessary in either direction should high loads be applied to any side of the framework.

These diagonal members were wrapped in thick covering to eliminate any bond between the steel and the concrete exterior wall.

A full-sized column section with a large column weldment and X-bracing was put in a 1,200,000 lb testing machine at Pittsburgh Testing Laboratory and pulled to destruction. Two 1-in. x 8-in. ASTM A514 X-bracing members were used in the test. One of them broke at 947,000 lbs. Sixty strain gages placed over the complete test piece gave incremental load readings throughout the destruction test. The recorded readings established were very close to the calculated design limits.

No date has been set for future expansion of the structure to its full design capacity of 24 stories. The building will be used jointly by Bell Telephone of Pennsylvania to relieve its expanding Western Pennsylvania requirements and American Telephone and Telegraph Company as headquarters for the overseas operation.

Steel Vierendeel Tops Office/Lab

Architects: E/4 Davies & Wolf Architects, Inc.
Cambridge, Massachusetts
Structural Engineer: Randall and Colcord
Boston, Massachusetts
General Contractor: The Carlson Corporation
Cochituate, Massachusetts

by M. Wyllis Bibbins

A hybrid-vierendeel roof truss system was the economic answer to the complicated functional requirements of the Honeywell, Inc., office-laboratory complex in Framingham, Mass. Employing the hybrid-vierendeel truss enabled architects Davies & Wolf and structural engineers Randall and Colcord to integrate a flexible mechanical system into the space required for a long-span structural system.

The inter-relationships which exist within this dual-purpose building were first studied by the architects through a series of interviews with company

M. Wyllis Bibbins, a principal of the firm E/4 Davies & Wolf Architects, Inc., was the Project Manager for the Honeywell complex.

employees. It was decided to provide for future growth by arranging the complex in a series of eight square growth modules. These modules together with an administrative building and the required parking and service facilities were arranged around existing features of the site (an old nursery) consisting of rock outcroppings, a pond, specimen trees and groves.

A further result of studying the company's needs was the realization that there was a need for frequent personal contact between engineers and technicians which required a close interweaving of office and laboratory space. The desired interweaving was achieved by arranging the laboratory space as a cross within each growth module, leaving quadrants in the corners for office

Six-ft truss depth in mechanical area allows maintenance personnel sufficient headroom to work without disrupting laboratory operations.

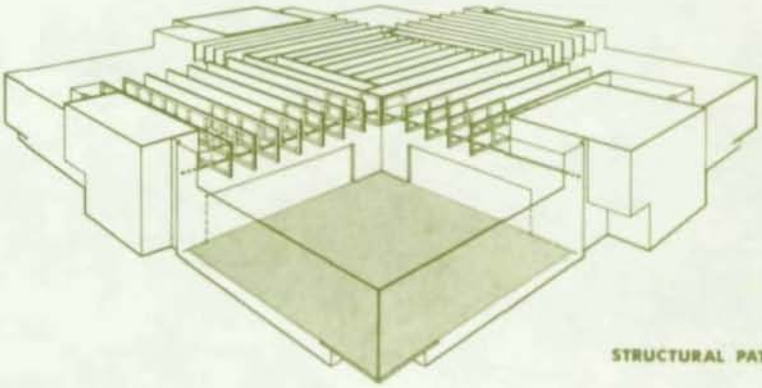
Site plan shows arrangement of eight modules.

use. Consequently, the laboratories serve as the linking elements between offices and between modules. This arrangement encourages engineer traffic to move through the laboratory areas and facilitates easy contact with the laboratory personnel.

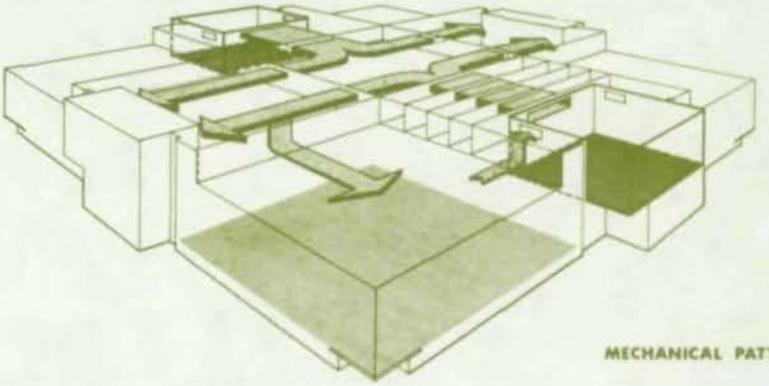
The hybrid-vierendeel trusses clear-span the laboratory areas and form overhead mechanical highways. Air-conditioning, heating, and ventilating ducts feed into the trusses from mechanical rooms at the ends of the buildings. Many of the ducts are large and this need for rectangular openings suggested the use of a vierendeel type of truss. However, a pure vierendeel truss required 50 percent more steel than a comparable pinned truss; to achieve economy it was decided not to provide rectangular panels throughout the entire truss, but only as many as needed to accommodate the large ducts with some room for flexibility and growth. The hybrid-vierendeel was planned with the middle three panels of the truss designed with moment-resisting joints and the balance of the panels containing diagonals and designed with pinned joints. All joints, however, both moment-resisting and pinned, are welded.

The truss depth is approximately 6 ft, making it possible for maintenance personnel to have sufficient headroom to move around within the truss area and make whatever mechanical and electrical modifications are required as changes are needed, without disrupting the laboratory operations below. The truss space also acts as a ductless return air plenum, so there is no ceiling in the laboratory space; this facilitates access to the mechanical services in the truss space as well. A tectum roof deck is used, which provides acoustic absorption in the space. Fluorescent lighting fixtures are mounted on the bottom of the trusses; the repetitive layout of the trusses on six-foot centers in the large partition-free laboratory space establishes a visual ceiling plane, which gives order to the appearance of all the various mechanical services threading through the trusses above. Three modules have been completed; others may be added one or more at a time as the space requirements of the owner demand.

Hybrid-vierendeel trusses clear span laboratory areas and form overhead mechanical highways.



STRUCTURAL PATTERNS

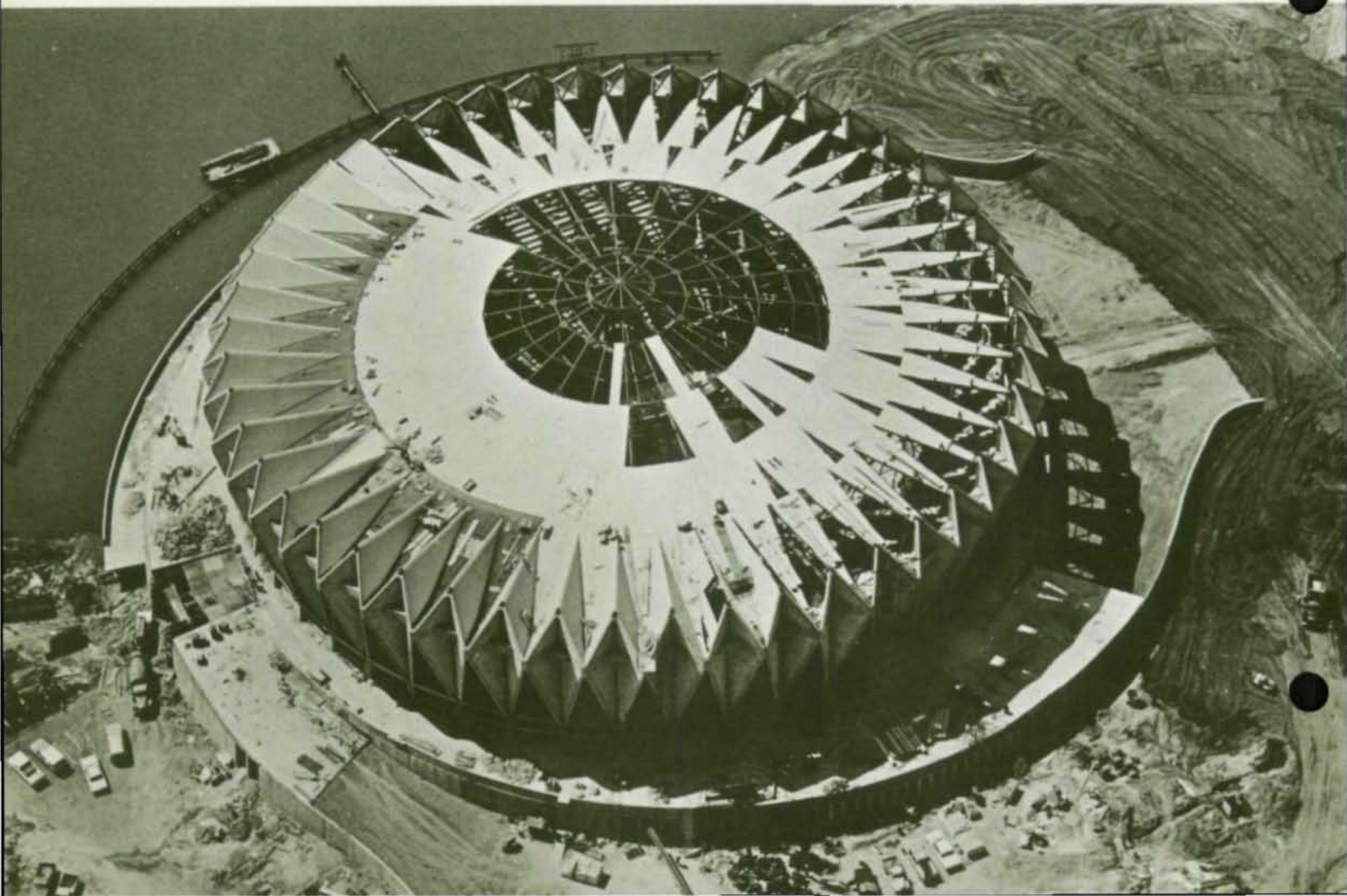


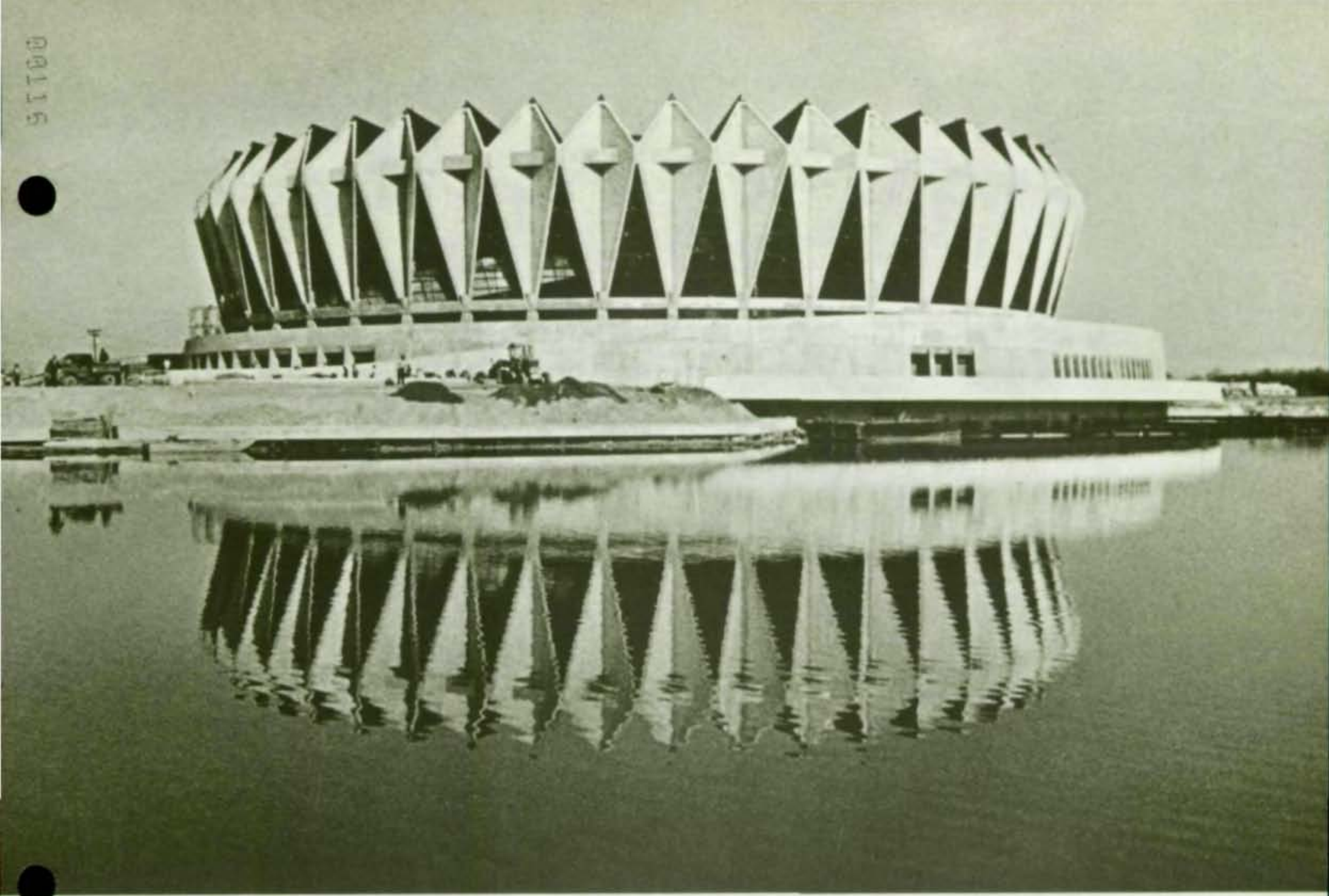
MECHANICAL PATTERNS

Schematic diagrams.

STEEL CROWNS THE HAMPTON ROADS COLISEUM

Architect: A. G. Odell, Jr. and Associates
Charlotte, N. C.
Structural Engineer: Severud-Perrone-Sturm-Colin-Bandel
New York, New York
General Contractor: McDevitt & Street Company
Charlotte, N. C.
Steel Fabricator: Bristol Steel & Iron Works, Inc.
Bristol, Va.





The design of the recently dedicated $6.8-million Hampton Roads Coliseum, in Hampton, Va., was inspired by the symbol of the Tidewater area — white sails. Exterior sail-shaped wall panels support a 400-ft dia. cable-suspended clear span roof.

The coliseum will accommodate conventions, trade shows, exhibitions, athletic events, musical productions, and other programs.

The building contains 246,160 sq ft of floor space on arena and concourse levels. The arena level includes a portable basketball court that can make way for a 200 x 85-ft ice rink. Lockers, dressing rooms, 2 exhibit areas, 4 meeting rooms, and a kitchen flank the arena area. The concourse level contains 24,248 sq ft of exhibit space. Capacities up to 11,000 can be easily handled.

Design Features

The impressive exterior was achieved with 96 diamond-shaped precast concrete panels weighing about 26 tons. The 60-ft high panels were paired to form 48 units around the perimeter. They are located on 7-ft high caps and are situated directly over the 48 exterior columns. The height of the structure is 85 ft.

A total of 7,344 ft of 2-in. dia. zinc-coated steel strand was used for the single layer roof. The 48 lengths of cable, each 153 ft long, were shipped coiled with open sockets at one end and Type 7 anchor sockets on the other.

About 340 tons of structural shapes and plates, primarily ASTM A572 steel, and approximately 22,000 ASTM A325 high-strength bolts were provided for the project.

Steel Erection

A cable and truss system was designed for the roof.

On the ground the open end sockets of the cable lengths were affixed to a tension ring. The steel plate ring is 16 in. wide, 4 in. thick, and has an outside diameter of 15 ft.

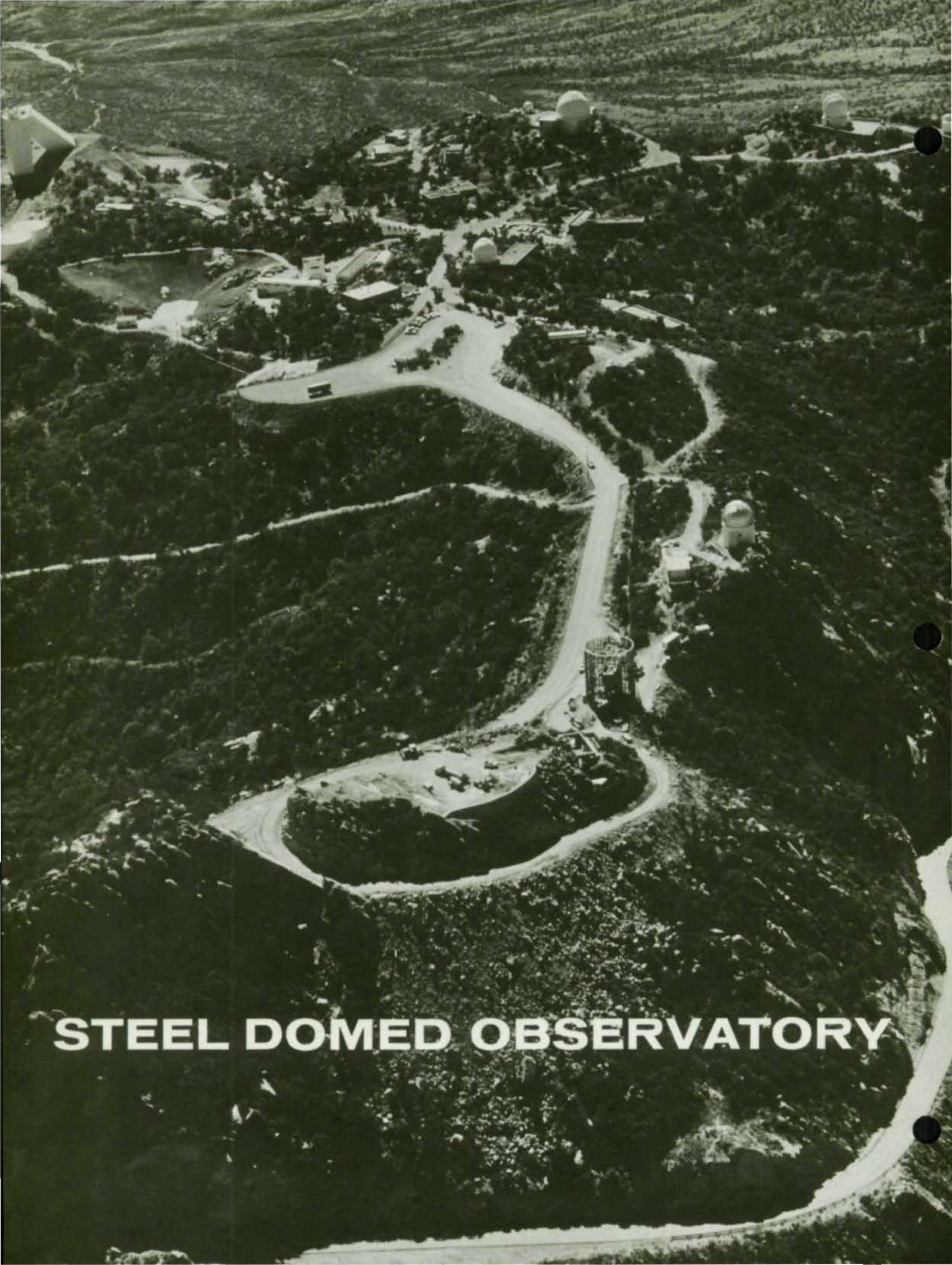
A crane then raised the tension ring to the height of the wall panels. The anchor-socketed ends were inserted one by one through weldments at the top of the wall panels and fastened with spanner nuts. After all 48 cables were in place, the tension ring was lowered to its free-floating position.

The cables sloped downward toward the center of the coliseum. To provide a roof pitch away from the center, the engineers designed three circular steel trusses connected by beams that were clamped atop the cables. The A572 steel truss members are primarily 10WF33.

The trusses solved the roof drainage problem and, combined with horizontally folded extensions of the wall panels, will also prevent roof flutter.

The compression ring is about 20 ft below the tops of the perimeter panels. The cast-in-place ring pierces the widest part of the panels, which is 16 ft.

The entire roof support system is exposed inside the coliseum, and the huge scoreboard is suspended from the tension ring. This system allows for unobstructed vision from any seat in the house.



STEEL DOMED OBSERVATORY

The University of Arizona's 90" Steward Observatory in Tucson had program requirements similar to any optical telescope observatory, and, as such, had many dictates which produced strong design considerations. A requirement that the enclosing structure respond quickly to the temperature changes and not hold heat for prolonged periods prompted the decision to use a thin, premanufactured corrugated type metal siding which could be painted white for maximum reflection. This material was backed up with a structural steel frame capable of carrying the weight and forces of the rotating dome, observation floors, and wind loads.

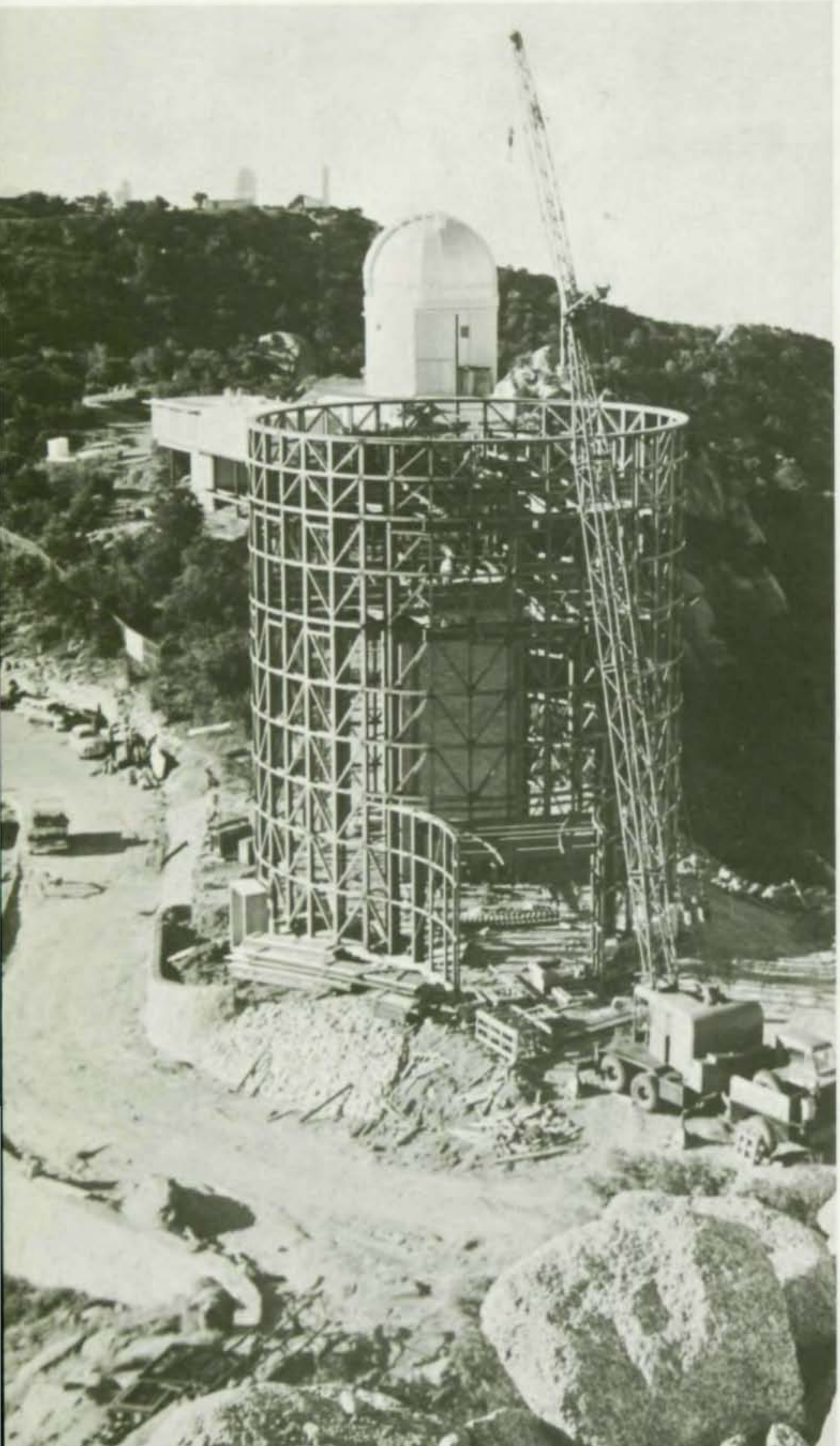
With a limited but adequate budget, and the distance of over 50 miles from the local construction market, with 10 miles of this up a very steep mountain road, an early decision to use steel as much as possible was made in order to achieve an economical design. All steel components were designed with a view to encouraging the contractor to use panelized construction of the structural framework. The contractor not only panelized the silo framing, but also the framing and skin surfaces of the observatory dome. The framing system selected for the observatory floors was in keeping with the original objectives, using a structural steel framing system to receive a composite floor system of corrugated steel decking form with a thin concrete top slab.

Analyzing The Problem

The architectural solution was a response to the programmed requirements of the telescope research. The original concept was a circular silo with two protrusions to house instrumentation from the telescope's coudé. (A coudé telescope is one in which the light path from the primary mirror is routed by means of secondary flat mirrors along the axes of the mount in such a manner that they are brought to a focus at various selected stationary points of observation or instrumentation. The points of observation are thus independent of the direction of orientation of the telescope.) This concept had two major drawbacks in that the isolated protrusions became almost impossible to solve within the limits of the vibration allowed, because of instrumentation to be placed within these areas.

An analysis of the problems produced the final solution which, in essence, folded open the silo to allow more areas for the coudé and allowed greater area for instrumentation within the silo. Folding the light beam down instead of sideways allowed the capability of providing mezzanine levels below the coudé. This was to substitute for the loss of the two protruding instrument areas. The entire solution was accepted and as the two protruding instrument areas originally programmed were areas that would not be put to immediate use, it was decided to make provisions only for these rooms below the coudé, as this future work could be performed without any complicated construction work at a later date. The folded opening at the front (main) truck entrance was necessitated by the program requirements for trucking delivery and handling of the various telescope and instrumentation components.

Steel framing construction allowed for the entire telescope installation through the interior of the observatory by merely increasing the hatch doors a foot in one direction and increasing the dome crane capacity three tons. The installation was accomplished without the use of an external (vehicular) crane, a savings of many thousands of dollars in comparison with the slight increase in cost. Because of the governing wind load, this change did not increase the size of the silo framing and only slightly increased the weight of the main dome framing members.



Cylindrical Dome Selected

The unusual shape of the observatory dome was a result of a study of two alternate solutions. First, the conventional concept of a spherical dome allowing for dome crane storage would require a much larger dome and this larger dome would, in turn, require a larger silo structure, or a complicated cantilevered engineering problem.

The second solution, as finally selected, was the use of a cylindrical dome which only had one technical drawback, the ability to provide permanent weathering on a flat top cylindrical dome unit. With the new exotic caulkings, this problem was solved satisfactorily. This cylindrical dome design had many other advantages over the conventional spherical design. Accessibility to all parts of the dome, crane, and upper surfaces for attachment of instrumentation was made extremely simple. Safety was improved and maintenance for the telescope was made easier.

Other Features

The telescope pier for the observatory was the only element that could not be constructed of structural steel. To overcome the high cost of a poured concrete structure for this pier, the structural engineer proposed a system of a double layer of concrete masonry units forming a 45-ft high lift grouted center core which was poured continuously in a single pour.

Entrance to the upper levels was designed for an elevator because of the extreme heights involved, and a standard manufactured metal spiral stair was used for an emergency staircase. The staircase, 40 ft high with three and one half spirals, is impressive, interesting, and almost adventurous to use.

Architect: William Wilde, AIA
Tucson, Arizona
Structural Engineer: Morris Self, P. E.
University of Florida
Gainesville, Florida
General Contractor/Steel Fabricator:
Allison Steel Manufacturing Co.
Phoenix, Arizona

Throughout the centuries, man and beast have struggled for control of environment. Civilization is evidence that man has an edge over beast. Occasionally, however, the beast upsets the pattern and man must eat crow — no mean accomplishment when the crow beats man to it.

We think readers of Modern Steel Construction will be amused by the following report of an actual, but unexpected, incident that preceded the AISC Prize Bridge ceremonies at the Queens Zoo-Aviary Pedestrian Bridge in Flushing Meadows-Corona Park, New York (see MSC, 3rd Quarter, 1969). The author is the 11-year-old daughter of AISC Regional Engineer Samuel H. Marcus, who arranged the awards presentation.

Crow Steals Scene at Prize Bridge Awards

by Ivy Dawn Marcus

My father is an engineer for the American Institute of Steel Construction, which is a trade association representing the steel fabricating industry.

Clarke and Rapuano, consulting engineers of New York City, designed the steel pedestrian bridge located within the Queens Zoo-Aviary at the Flushing Meadows-Corona Park. The Triborough Bridge and Tunnel Authority built the bridge for the New York City Department of Parks. The bridge won an AISC award for being one of the most beautiful bridges in the country.

My father was supposed to present the award to Robert Moses, consultant to the T. B. T. A., Dr. William J. Ronan, Chairman of the Metropolitan Transit Authority, August Heckscher, Commissioner of the Department of Parks, Gilmore D. Clarke, President of Clarke and Rapuano, and Edward Simpson, President of NAB Construction Corporation.

Little was my father to know that later in the day he was going to be attacked. It was the day before he was to present the awards and he went to the aviary with two other men. They were discussing plans for the ceremony, when suddenly a crow swept down from a tree aiming for my father like a vampire. It clawed his head, then went away. A second time it swung at him and he used his umbrella to push it away. My father put his hand to his head and felt something warm and wet below his bald spot. It was blood —the crow had scraped his head! A third time it tried to strike him, but my father pushed him away and walked out of the aviary, home to my mother.

When he returned home, my mother described him as if he was going to die. My father was frantic. They went to a doctor. When the doctor heard what happened he was hysterical. The doctor thought his case the day before was funny, when a lady who was watching the Merv Griffin Show had a toaster fall on her head. But a man being attacked by a crow really made his day. The doctor gave my father a tetanus shot and his adventure was over.

The moral of the story is: Men with bald spots should not go to aviaries. P. S. My father is O. K. today and so is Agnes, the crow.

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