VOLUME XV / NUMBER 1 / FIRST QUARTER 1975

MODERN STEEL CONSTRUCTION

1

2

3

MODERN STEEL CONSTRUCTION

Published by

American Institute of Steel Construction

1221 Avenue of the Americas
New York, N.Y. 10020

VOLUME XV / NUMBER 1 / FIRST QUARTER 1975

CONTENTS

1975 T. R. HIGGINS LECTURESHIP AWARD

Mr. E. Alfred Picardi has been named recipient of the Fifth Annual T. R. Higgins Lectureship Award. Mr. Picardi was chosen to receive the $2,000 award for his contribution to the fund of engineering knowledge as the author of "Structural System—Standard Oil of Indiana Building" (ASCE Journal of the Structural Division, April 1973).

The award will be presented at the 1975 National Engineering Conference banquet on Thursday evening, May 1, in St. Louis.

1975 PRIZE BRIDGE COMPETITION

Entries are invited for the 47th Annual Prize Bridge Competition to select the most beautiful steel bridges opened to traffic during the calendar year 1974.

The members of the 1975 Prize Bridge Jury are:

Ruben N. Bergendoff, F.ASCE *Howard, Needles, Tammen & Bergendoff, Kansas City, Missouri*

Arthur J. Fox, Jr., F.ASCE *President-elect, American Society of Civil Engineers; Editor, Engineering News-Record, New York, New York*

John M. Hayes, F.ASCE *Professor, School of Civil Engineering, Purdue University, West Lafayette, Indiana*

William N. Holway, F.ASCE *Executive Vice-President, Benham-Blair & Affiliates, Inc., Tulsa, Oklahoma*

Nelson C. Jones, F.ASCE *Assistant Deputy Director (Retired), Michigan Highway Department, East Lansing, Michigan*

Entries must be postmarked prior to May 31, 1975 and addressed to the Awards Committee, American Institute of Steel Construction, 1221 Avenue of the Americas, New York, New York 10020.

1975 FELLOWSHIP AWARDS

Four engineering students have been awarded $3,500 fellowships in the 13th Annual Fellowship Awards Program. The program is designed to encourage expertise in the creative use of fabricated structural steel.

Bruce C. Barrett *Brigham Young University*

Aldo F. Colandrea *University of Detroit*

Daniel B. Goetschel *California State University at Northridge*

Gary R. Kuhn *Arizona State University*

To enter
your subscription to the
AISC ENGINEERING JOURNAL
detach and return
the attached envelope
today

Please enter my subscription to the AISC ENGINEERING JOURNAL

U. S. and Canada

☐ 1 Year $5.00
☐ 3 Years $12.50

***Other Countries**

☐ 1 Year $ 6.00
☐ 3 Years $15.00

*Payable in U. S. Funds

Payment **must** be enclosed with this order.
Make checks payable to AISC.

NAME (please print)

FIRM OR AFFILIATION

ADDRESS

CITY STATE ZIP CODE

Profession or occupation:

☐ Engineer ☐ Architect ☐ Educator ☐ Other ____________

If you
design steel
structures....

you need
the AISC
ENGINEERING
JOURNAL

FROM ______________________

ADDRESS ______________________

CITY ____________ STATE/COUNTRY ____________ ZIP ____________

FIRST CLASS
PERMIT No. 62038
NEW YORK, N. Y.

BUSINESS REPLY MAIL
NO POSTAGE STAMP NECESSARY IF MAILED IN THE UNITED STATES

— POSTAGE WILL BE PAID BY —

AMERICAN INSTITUTE OF STEEL CONSTRUCTION
1221 Avenue of the Americas
New York, New York 10020

Attention: Paul R. Johnson

by James R. Fincher

ATLANTA'S NEW COLISEUM

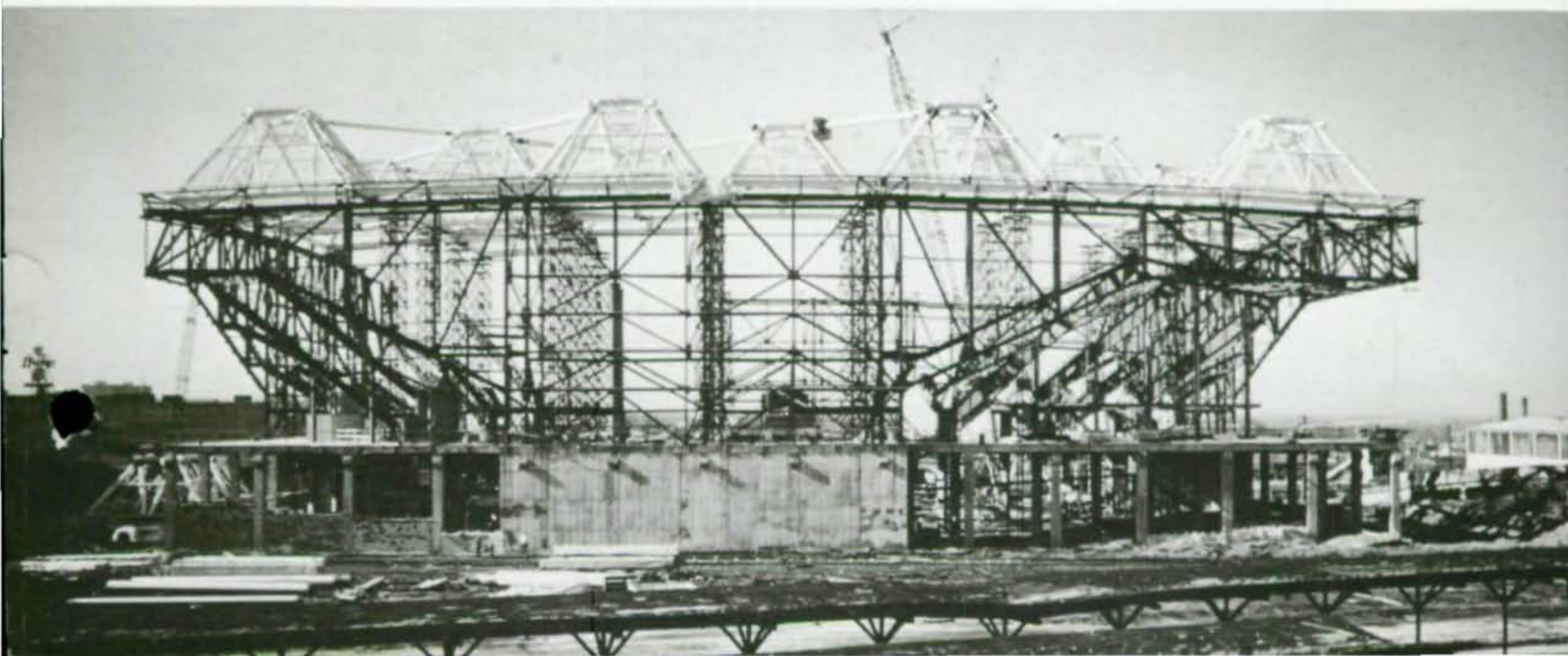
The challenge of designing an economical structure to clear-span a square 350 ft on a side, resting on walls 100 ft high, is not to be taken lightly. Add to this a restriction that two-way trusses are not acceptable because of appearance and that the facility must be operating in less than two years, and the problem becomes formidable.

Mr. Fincher is President, Quo Modo, Inc. and Sr. Associate, Prybylowski and Gravino, Inc., Atlanta, Ga., structural engineers for this project.

Basically the problem was to provide the design for a sports coliseum — the Omni in Atlanta, Ga. — to accommodate 16,000 seats for professional hockey and basketball games, as well as to house other activities such as ice shows, concerts, and conventions. The location of the site, hemmed in by vehicular viaducts and railroads, was a strong determinant for a square building configuration. The architect centered the seating "bowl" on the diagonals of the 350-ft square plan to provide advantageous sight lines for all seats. This seating arrangement set the plan size as well as the form and height of the wall trusses.

Roof Structure

Studies of various structural schemes consistently revealed that a two-way truss roof system would be highly de-



The Omni Coliseum, Atlanta, Ga., can accommodate up to 16,000 people.

sirable. However, the owner required that the structure not only provide a dramatic ceiling to the space, but that the roof be visually interesting and attractive when viewed from the tall office buildings surrounding the site. Considering the owner's restrictions, as well as the need for economy, the required solution appeared to be one of devising a steel two-way truss that did not look like a two-way truss.

The final design solution — one that satisfied both the aesthetic and economic criteria of the owner — was a unique space truss conceived as an orthogonal intersection of trapezoidal folded plates connected by diagonal upper chord members.

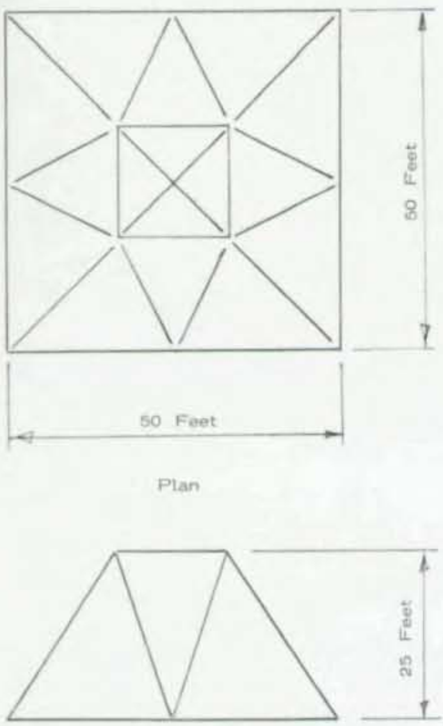
To visualize the resulting structure, consider a chess board having only seven rather than eight squares to a side. Begin at one corner square and place on every alternate square (say black) a truncated pyramid about half as high as the plan dimension of the black squares. Leave the red squares as flat surfaces, then connect the top corners of adjoining truncated pyramids with diagonal members (see the schematic drawing of the roof plan and elevation).

Because the roof was required to rest on approximately 100-ft high walls, erection was a major consideration. Lifting a small number of completed large substructures was selected as the erection scheme. The truncated pyramids, called "pods," were ideally suited for such a scheme; framed with steel, they were light in spite of their large overall dimensions (50-ft square at the base, 25-ft high).

The schematic drawing of a typical pod shows the arrangement of the steel framing. Although steel sizes varied with load requirements, all base members were wide flange sections, while diagonal members and braces were pipe sections.

As in any space structure, the connections make the difference in a practical, economical, and safe structure. Great attention was paid to the connection details and many comparative

POD FRAMING SCHEMATIC



Plan

Elevation

studies were made. Fully welded pods, partially shop fabricated and then field assembled by welding in a special jig, was the only scheme that satisfied all requirements. For example, a typical bolted joint would have required 300 1¼-in. diam. A490 bolts and would have undoubtedly required expensive field reaming because of the thick materials being joined.

Some welding problems were encountered, but they were solved by attention to normal good practice of proper joint design, good joint preparation and fit-up, and competent, qualified welders.

Both the pods and the squares of flat roof deck were covered with a weathering steel exterior finish.

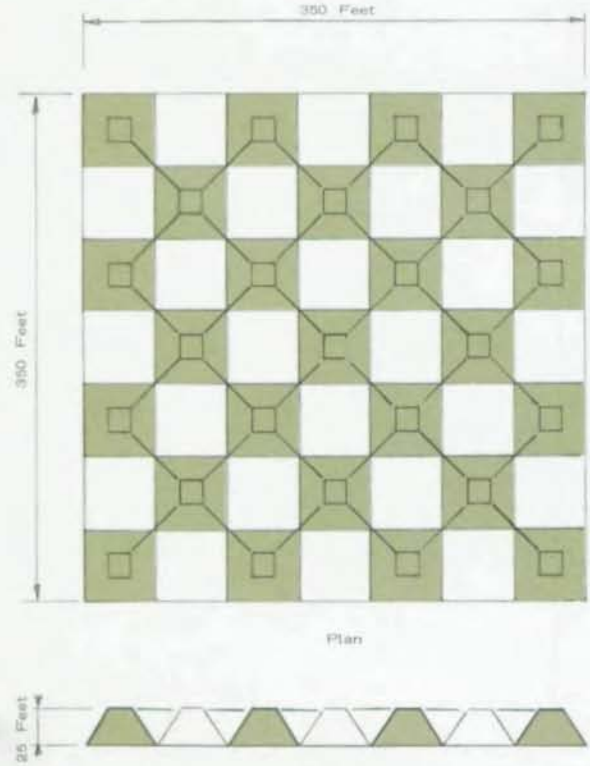
Wall Trusses

The form of the supporting wall trusses, approximately 100 ft high, was determined by the intersection of the seating bowl (see the schematic diagram of the wall trusses). The combination of this strong central tower section and cantilevered ends was then used to advantage both structurally and architecturally. The tower was ideal for resisting wind forces, while the normal tendency of the cantilevers to deflect applied reverse forces to the roof structure.

Architecturally, in the corners beneath the seating bowl, four glass enclosed lobbies were formed by the 100- and 150-ft cantilevers of the wall trusses, the 45-ft high glass curtain walls being framed with light steel trusses to resist wind forces. The exterior of the building was sheathed in weathering steel with feature strips applied to the surface identifying the structural frame of the wall truss.

The completed structure satisfied the initial criteria of an economical design having pleasing appearance both from the outside and from the spectator's viewpoint. The structural weight of the roof was approximately 16 psf, which compared most favorably with a two-way truss system at about 23 psf. Thus, a higher in-place unit cost was offset and the "premium" for appearance was negligible.

ROOF SCHEMATIC



Plan

Elevation

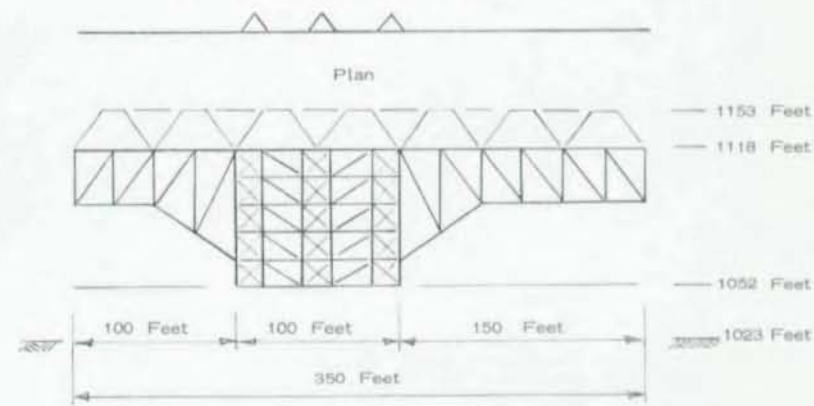
Architect:
Thompson, Ventulett & Stainback Inc.
Atlanta, Ga.

Structural Engineer:
Prybylowski and Gravino, Inc.
Atlanta, Ga.

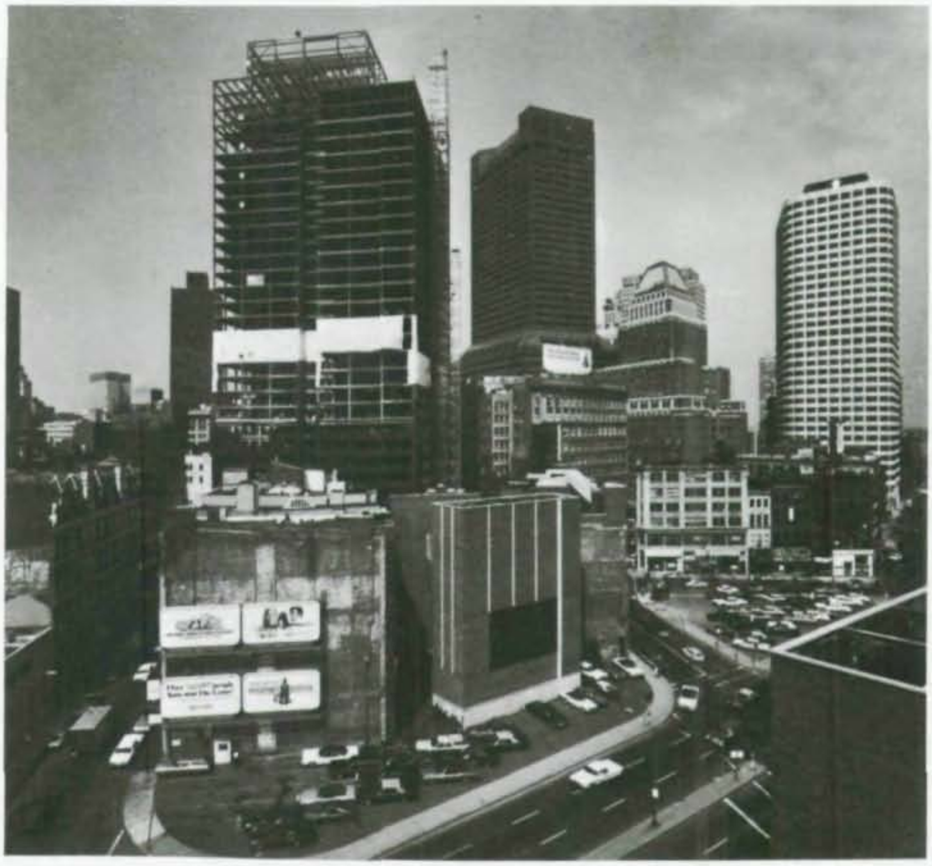
General Contractor:
Ira H. Hardin Company
Atlanta, Ga.

Steel Fabricators:
Mississippi Valley Structural Steel
Division of Debron Corporation
Chattanooga, Tenn.
Steel Inc., Scottsdale, Ga.

WALL TRUSS SCHEMATIC



Elevation



A LIGHTWEIGHT HI RISE

by Stanley Goldstein

Major high-rise building requiring 36 percent less steel than any other structure of comparable size was achieved through structural design that was consciously directed toward economy. Use of high-strength steel, composite design and lateral stability systems designed and refined by analytical modeling with the aid of a computer were the primary methods of accomplishing this remarkable savings in material.

Located in windy downtown Boston, the 100 Summer Street tower (Blue Cross/Blue Shield is a major tenant), which utilizes only 12¼ lbs of structural steel per sq ft, reflects a major breakthrough in high-rise framing economy. This exceptionally low weight was determined by the developer, Cabot, Cabot & Forbes, based on the actual total steel tonnage supplied.

Overall plan dimensions are 180 ft x 240 ft and the structure rises 33 stories (420 ft). A 40 ft x 90 ft central core contains elevator, duct, pipe and electrical shafts as well as a lobby, toilets, and stairways. Deep notches intruding on both north and south facades form a critical part of the massing of the building, which in combination with the bronze-tinted aluminum skin won approval of the vigilant Boston Redevelopment Authority.

Since the tenant for the lower half of the building required electrifiable floors, a 3-in. deep blended metal deck was provided for these floors, while 1½-in. deep metal deck was used elsewhere. All metal deck is composite and topped with a minimum 3¼-in. of lightweight concrete, achieving a two-hour fire rating without spray fireproofing.

Structural requirements for this building are not unusual. Mechanical rooms are located at the 9th, 27th, and 33rd levels, and a computer floor at the 11th level has a 150 psf live load. There are provisions for an offstreet truck loading space and a cafeteria is located on the second level. Roofs at

---

Mr. Goldstein, consulting structural engineer on this project, is partner-in-charge of LeMessurier Associates/SCI's New York office.

the 29th and 32nd levels have been designed to support paving and a 100 psf live load. In addition to wind loads, the Boston building code requires design for Zone 2 seismic loads.

Economical Structural Design

The structural design was consciously directed toward economy by considering the system as having three primary components: columns, lateral stability systems, and framing.

Column economy is achieved by using high strength steel (F_y=50 ksi), by eliminating as many columns as possible from the lateral stability system (allowing them to be designed for direct load only), and by providing a trussed lateral stability system that enabled the designer to use the effective length factor K=1.

Lateral Stability System

Due to the combination of wind and seismic loads, the achievement of lateral stability system economy appeared at first to be somewhat difficult, since wind forces are usually best resisted by stiff members while seismic forces are often best resisted by more flexible (ductile) systems. In total, the system must provide the following:

1) Stability against overturning
2) Strength against lateral forces
3) Strength and stiffness to brace the column system
4) Stiffness against lateral forces
5) Torsional strength and rigidity

Bracing trusses were installed in the core between elevator, duct and stairway shafts. East-west diagonals were spread apart to permit access to elevator lobbies. As low-rise and mid-rise elevator shafts terminate at the 16th and 24th levels, truss lengths decrease accordingly. The bracing trusses are complemented by moment-connected portal frames located in the east and west facades and along column lines 5 and 11.

These portal frames serve several purposes: (a) they provide the reserve ductile strength required by earthquake regulations; (b) they augment the trusses in resisting wind and earthquake forces, particularly at the top of the building, greatly reducing the overturning moment on the trusses; and (c) they provide great torsional strength and rigidity at no additional cost, since they are located far from the centroid.

Steel Framing

Framing economy is achieved by combining high-strength steel and composite design with a small typical bay size of 20 ft x 25 ft. Composite design of girders also provided more clear space for ducts, permitting a 12 ft-0 in. story height to be maintained. Since shear studs were placed on girders only, metal deck was spread apart at the girders to permit the use of ⅞-in. diameter studs rather than the less efficient ¾-in. studs required when welding through metal deck.

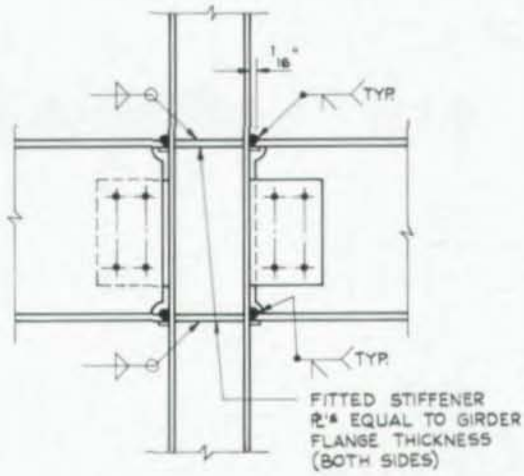
Almost all of the steel used for this project was ASTM A572, grade 50. Exceptions were the use of A36 for portal frames and A588 for thick plates. Shop connections were welded, while field connections were high-strength bolted. Portal frame columns were spliced 3 ft-6 in. above the floor to eliminate the need for special moment connections.

Computer Design

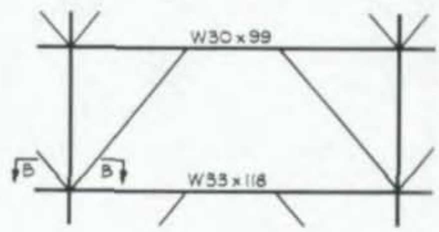
Lateral stability systems for the building were designed with the aid of an *IBM 1130 computer. In order to* obtain maximum understanding and control of the linked truss-portal systems, analytical modeling, both with respect to the number of stories and the number of elements, was performed. Curves of acceptable lateral displacements were developed as criteria for each direction. Preliminary steel sizes were selected for the entire lateral stability system.

The system was then analyzed by the computer and its deflections compared with the criteria, and the steel members were adjusted accordingly. The computer calculated influence lines so that optimum efficiency could be obtained. This process (called "tuning") was repeated more than 10 times in each direction until the optimum was selected.

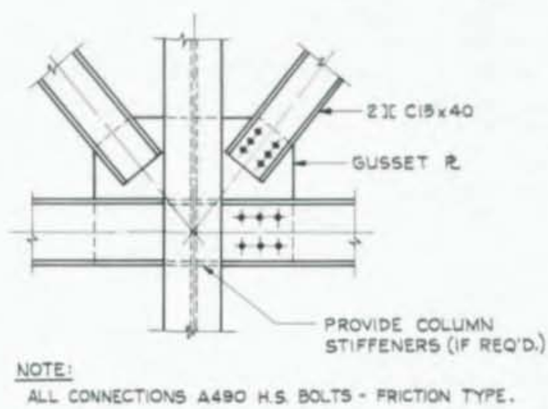
The resulting structure required only 61 percent as much steel as is usually required for a 33-story structure. To the best knowledge of the structural designers of this building, the lowest previous weight per sq ft documented for a building of this size was 19.1 psf (see *Modern Steel Construction*, First Quarter, 1972). Fabrication and erection were normal, so that the owner reaped the full benefit of the dollar value of the material saved, besides the value of the lower story height required.



Typical portal frame moment connection



SECT. A-A
Elevation—wind truss



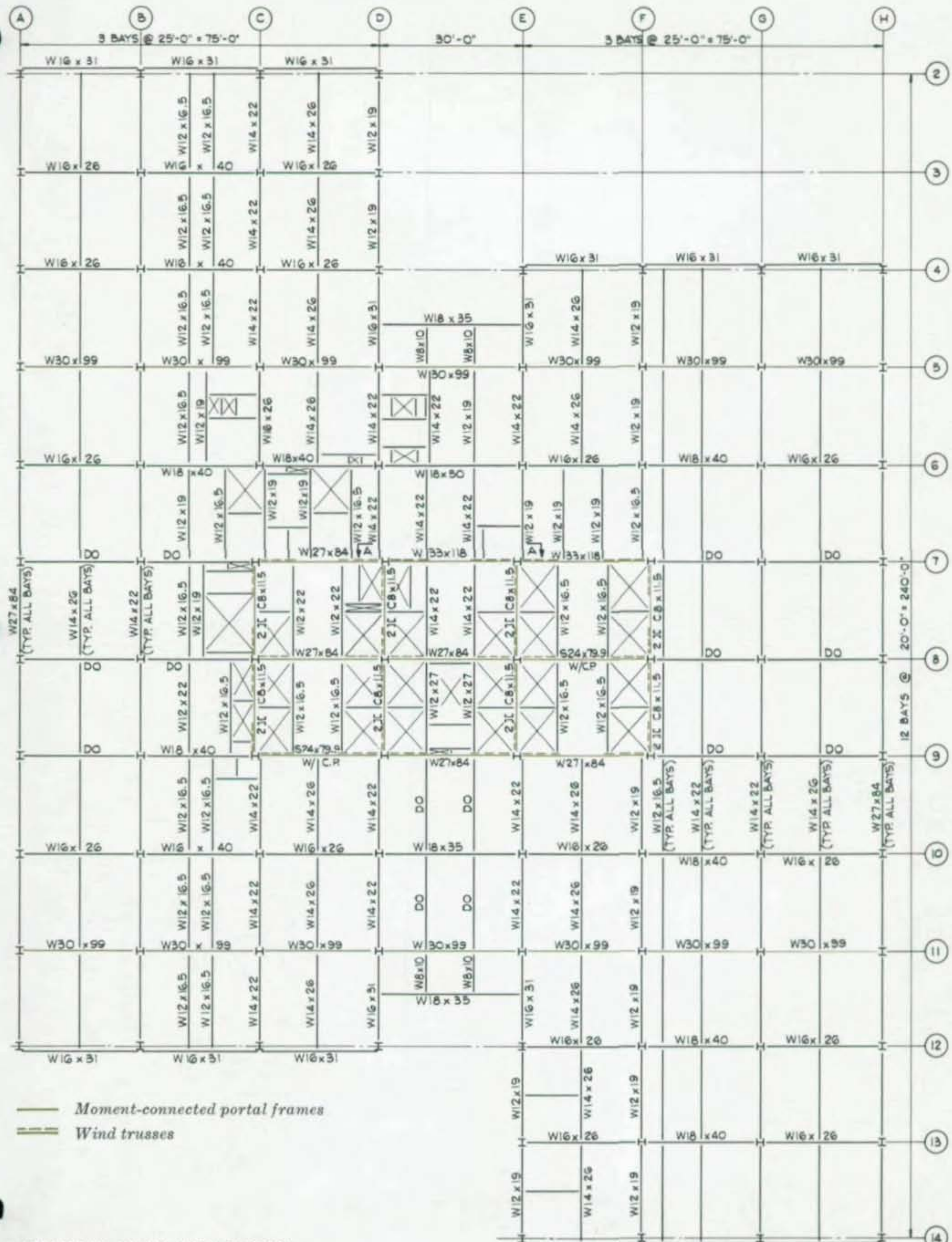
SECT. B-B
Wind truss detail

Architect:
Welton Becket and Associates
New York, N.Y.

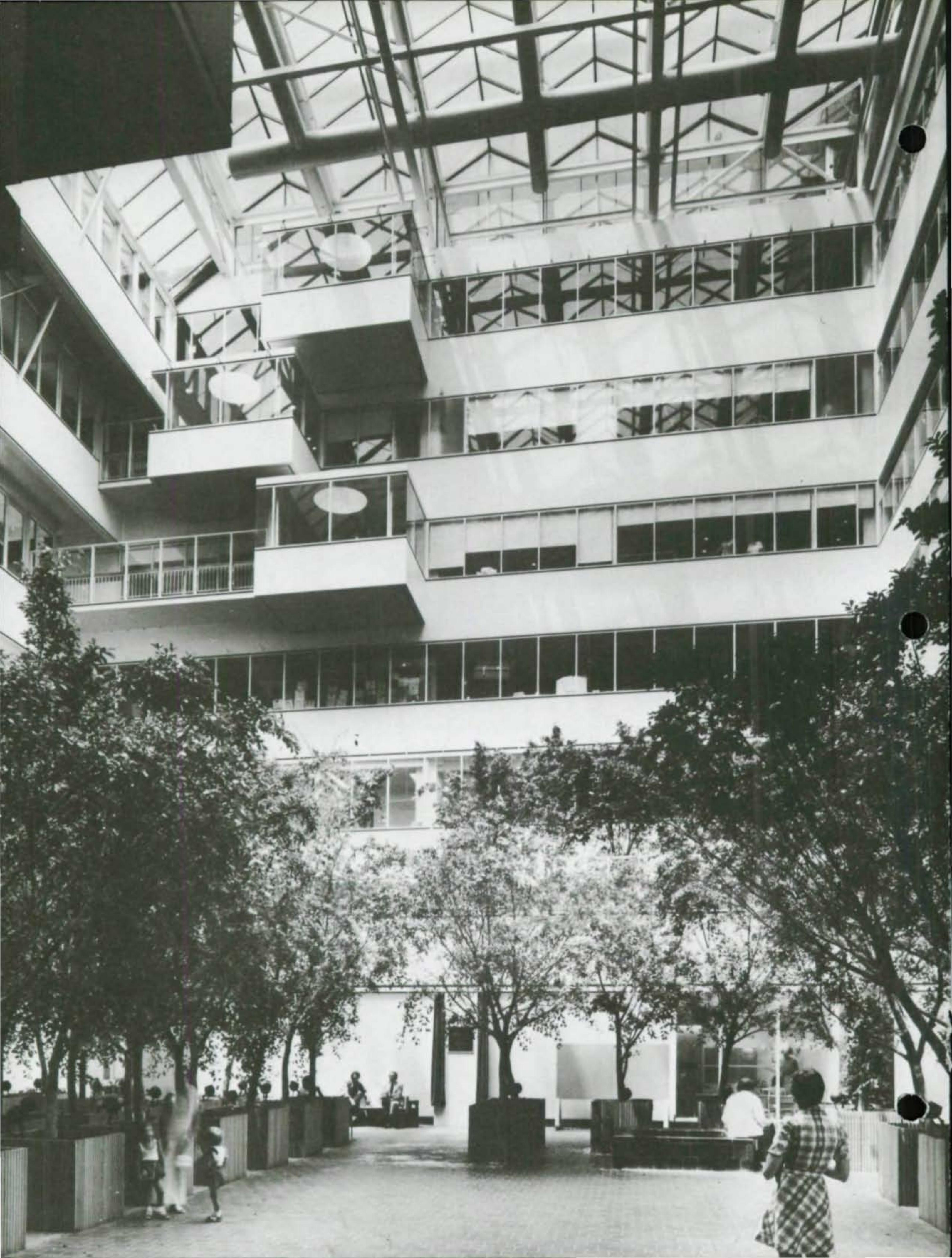
Structural Engineer:
LeMessurier Associates/SCI
New York, N.Y.

General Contractor:
Aberthaw Construction Company
Boston, Mass.

Steel Fabricator:
Harris Structural Steel Co., Inc.
Piscataway, N. J.



FRAMING PLAN (8TH FLOOR)



A CHILDREN'S HOSPITAL in Philadelphia

by William J. H. Hough, Jr., AIA

A nine-story landscaped central courtyard, topped with a transparent roof, dominates the Children's Hospital of Philadelphia, an all-comprehensive facility that dually serves as a major research center and as the pediatric teaching unit for the University of Pennsylvania School of Medicine.

Balconies at all nine levels surrounding the 100 ft x 100 ft court provide play, lounge, and staff areas. Half of all patient rooms and all dining areas also enjoy views into the court. The courtyard acts as a return air plenum for the Hospital's unique energy reclaim system.

Energy Reclaim System

Children's Hospital is the first major hospital in the country to be heated and cooled by an energy reclaim system

Mr. Hough is a partner of Harbeson Hough Livingston & Larson, Architects and Planners, Philadelphia, Pa.

that recycles heat from people, equipment and solar energy. The system also minimizes the escape of pollutants into the environment, using the maximum in previously wasted natural energy, purifying polluted air, and recycling heat for reuse.

General Features

The new 900,000 sq ft structure, with provision for expansion, serves patient care, medical education, and research functions. Two floors below grade provide parking space for 400 cars as well as for ancilliary and support services. The ground floor includes a chapel, branch bank, post office, gift shop, sidewalk cafe, and an auditorium seating 335 people. The out-patient department, located on the first three floors, accommodates 200,000 visitors annually. The roof contains a helipad serving the emergency transport system.

Research facilities are arranged on a modular system so that they can be adapted to shifting needs. Flexibility is provided by mechanical corridors which supply all services from electricity to oxygen, and each laboratory is able to simply "plug" into the needed services.

The interiors are uniquely designed for children, from furniture scale and primary colors to the provision for play programs, school facilities and procedures and techniques adapted to the special tolerances of infant, child, and adolescent. The hospital has an informal domestic atmosphere with low ceilings and bright graphics.

Steel Framing

Steel was chosen as the basic structural material because it met the following criteria: ease of acceptance of extensive mechanical and electrical systems, adaptibility to future additions and alterations, compatibility with architectural and first costs, and the many and varied programmatic demands developed by the owner. The subsequent use of steel decking with dovetailed ribbing also relates well to these criteria.

The basic structure is a computer-designed steel frame utilizing a 24-ft x 48-ft bay with infill beams also chosen by plastic design methods. The principal connections were welded for continuity. The use of steel was entirely consistent with the architects' desire to clad the building in lightweight curtain wall materials and to expose the structure wherever possible for economic and aesthetic reasons, as well as to provide fascinating eye-appeal for the young occupants of the building, their parents, and the staff.

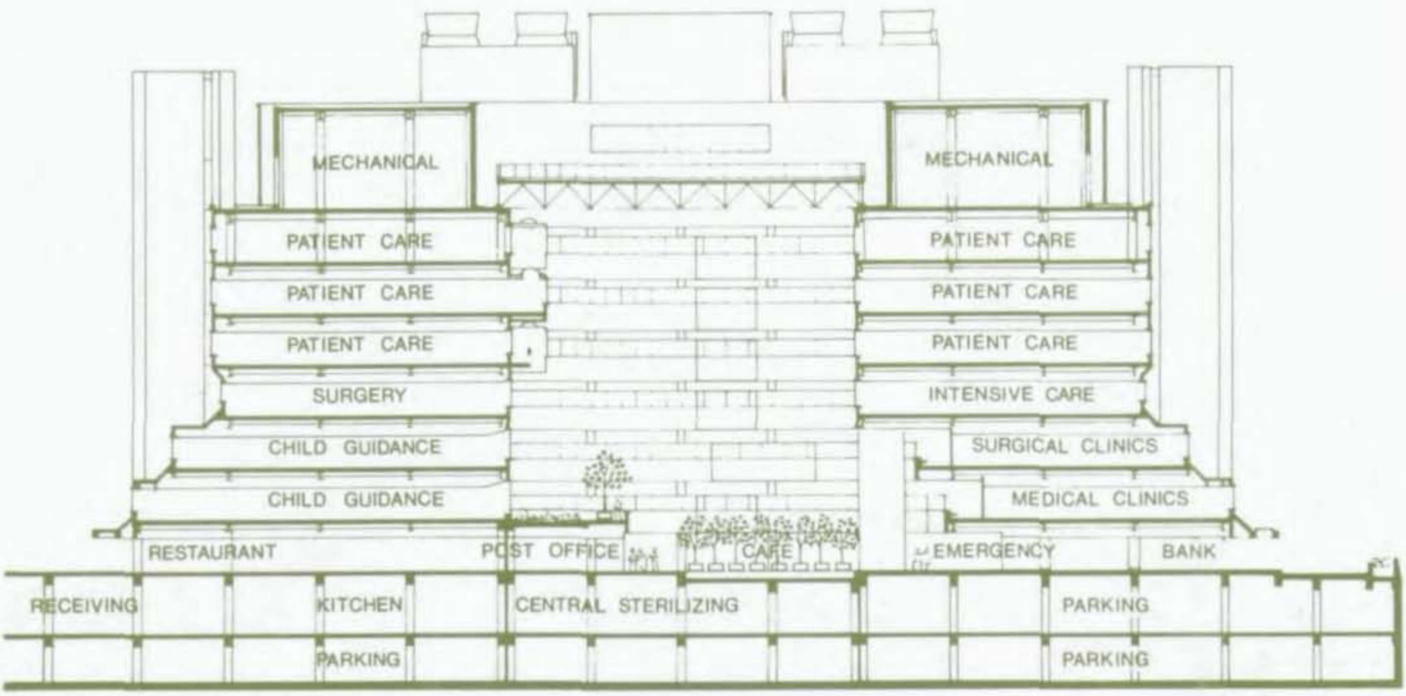
The large and carefully exposed steel trusses in the nine-story central court, the frank expression of tension members with their turnbuckles and other fittings for the hanging balconies within the court, and the extensive use of open steel grating in the five-story mechanical corridor demonstrate the aesthetic and economic benefits which can be derived from the use of steel.

Architects:
Harbeson Hough Livingston & Larson
William A. Amenta
Associated Architects
Philadelphia, Pa.

Structural Engineer:
A. W. Lookup Company
Philadelphia, Pa.

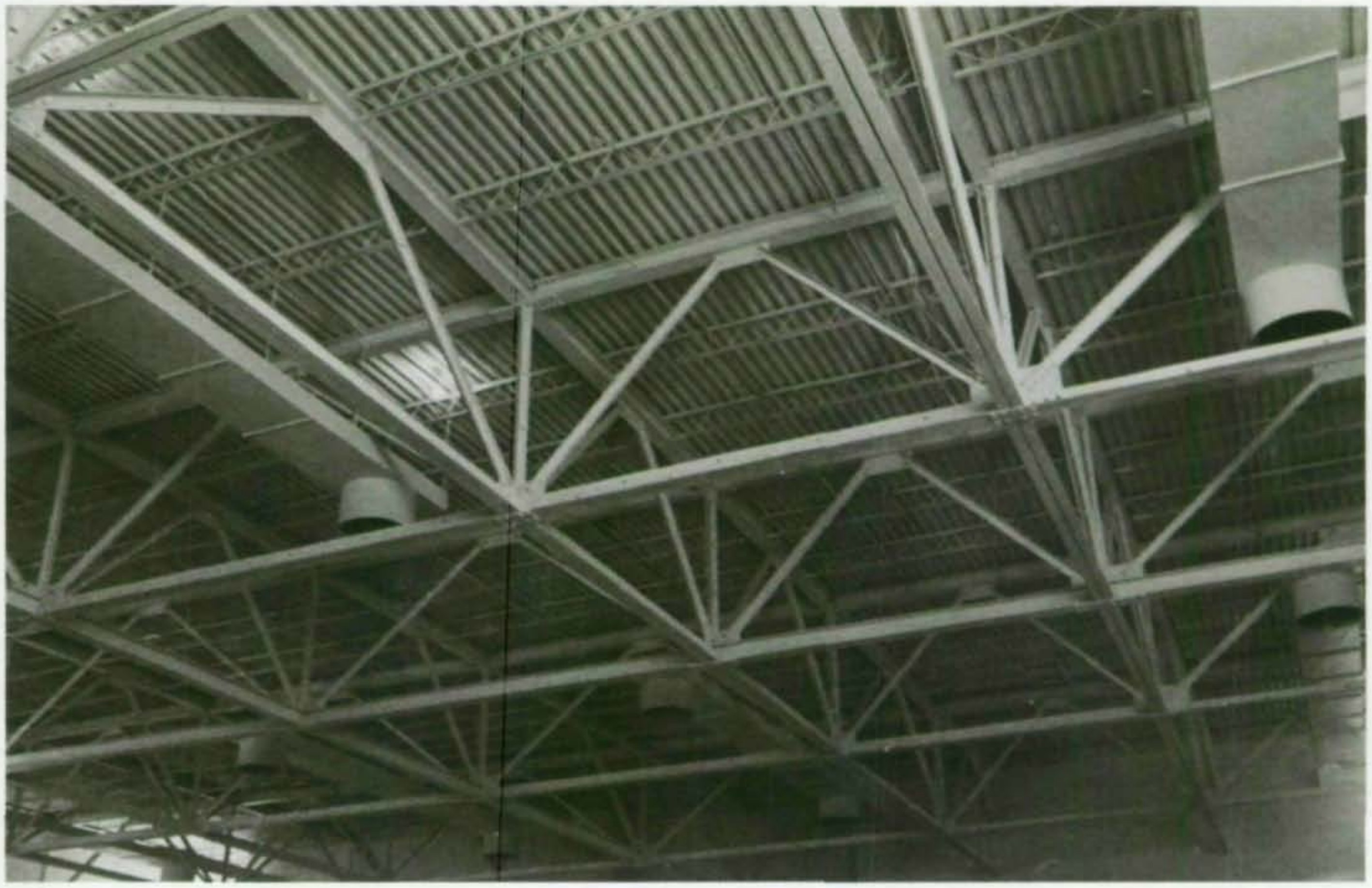
General Contractor:
Baltimore Contractors, Inc.
Baltimore, Md.

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



THE CHILDREN'S HOSPITAL OF PHILADELPHIA

NORTH SECTION



Two-Way Trusses for a HIGH SCHOOL GYMNASIUM

by Frederick M. Law

Architect:
Arthur Rigolo
Clifton, N. J.
Structural Engineer:
Kellerman & Dragnett, Inc.
Little Falls, N. J.
General Contractor:
Senia Construction Company
Franklin Lakes, N. J.

Perhaps the clearest expression of the structural engineer's art is the roof framing system of a modern high school gymnasium. Invariably, this completely visible structural system is one of the dominant elements in the total composition.

Although a number of different structural systems have been used to frame gymnasium roofs in recent years, the simple roof truss remains the one most often selected because of its high degree of structural efficiency. However, this efficiency is accompanied by an unattractive maze of lateral bracing between roof trusses. Fortunately, a solution does exist that meets with aesthetic approval and proves more efficient as well—the two-way truss system.

Dr. Law, Chairman of the Department of Civil Engineering, Southeastern Massachusetts University, North Dartmouth, Mass., served as Structural Consultant with Kellerman & Dragnett, Inc., the structural engineers for this project.

Two-Way Truss System

The two-way truss system carries the load in two directions instead of one. This results in smaller member sizes and a lighter roof system. In addition, no lateral bracing between trusses is required, thus achieving even more structural efficiency. Further, the trusses express a strong, clear statement of structural form which has universal visual appeal.

This combination of structural efficiency and clarity of the structural form led the designers to select two-way trusses to frame the roof of the Lakeland Regional High School Annex Gymnasium, Wanaque, N. J.

To meet the criterion of dividing the gymnasium into separate use areas by means of sliding doors, one truss was placed along the centerline of the roof in both directions. Space requirements in adjacent areas of the annex dictated the specific column locations. Additionally, the maximum span of the metal deck (and hence the spacing of the joists) led to the symmetrical, but uneven truss spacing.

Analysis of System

Unfortunately, the analysis of a two-way truss system is not as simple as the clarity of its form might suggest. Such systems are always statically indeterminate, usually to a rather high degree. Therefore, an exact analysis requires the formulation and solution of a series of simultaneous equations — a time-consuming and complex procedure. It is probably this computational difficulty, more than anything else, which has discouraged the use of two-way truss systems.

In an attempt to overcome this difficulty, it was assumed that the system was statically determinate for the preliminary design. The total load on each tributary roof area at each truss intersection was divided equally between each of the intersecting trusses.

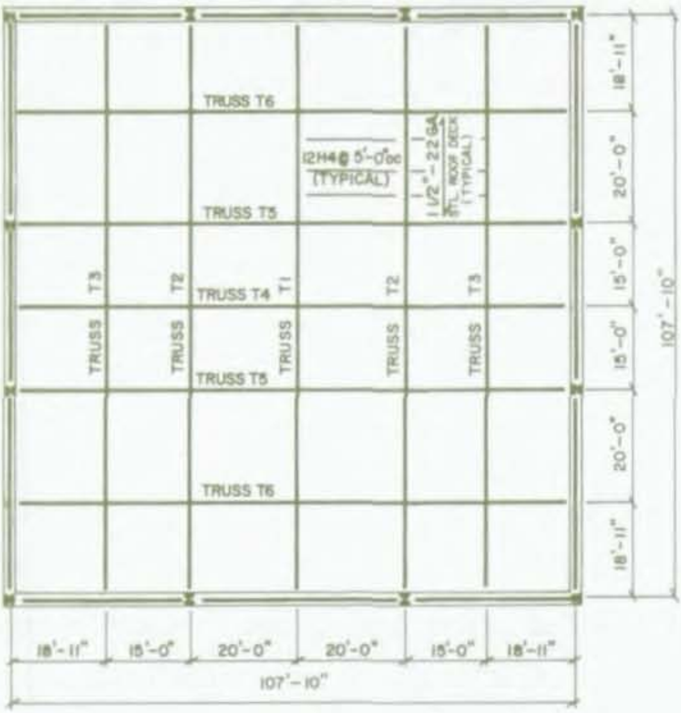
Admittedly, this assumption in no way attempts to consider the relative stiffnesses of the trusses. However, it is believed that the simplicity of the approach far outweighs any minimal loss in theoretical accuracy. Although it was not considered to be absolutely necessary by the author, a detailed analysis was made of the two-way truss system using the classical method of consistent deflections. The resulting member sizes for both analyses for each truss are indicated in Table 1 for comparison.

The method of consistent deflections requires: a) the formulation of deflection equations for each truss at each point of intersection in terms of the applied loads and the unknown interaction forces between trusses; b) the formulation of the compatibility equations which indicated that the deflection at each point of intersection is the same, whether approached from one direction or from the other; c) the formulation of the equilibrium equations for the system; and d) the simultaneous solution of the set of equilibrium and compatibility equations.

Taking advantage of the symmetries existing in the gymnasium, there were a total of 13 unknown deflections and 13 unknown interaction forces between intersecting trusses requiring the formulation of a series of 13 simultaneous equations. As formulated, they consisted of four equations of equilibrium and nine compatibility equations. The nine compatibility equations comprised combinations of deflections determined from a series of 22 separate deflection equations. A matrix inversion routine and an IBM 1620 computer solved the 13 simultaneous equations for the unknown interaction forces.

The finite difference method of analysis, a method frequently recommended for two-way truss systems, was considered for the analysis of the gymnasium roof. However, in order to gain advantage of ease in equation formulation, which the finite difference method yields, a flexural grid with approximately equal increments is required. In the case of the gymnasium roof, as with most real structures, the truss spacing was not equal in either direction, nor was the pattern the same in the two directions. Therefore, the finite difference method offered no advantage.

It is estimated that the use of the two-way truss system resulted in approximately a 20 percent savings of structural steel compared with the use of the simple one-way truss and associated lateral bracing system. Further, for this type of relatively small structure, subjected to ordinary loads, the simple approximate analysis performed appears to lead to member sizes not differing significantly from the sizes determined by the more time-consuming exact analysis.



ROOF FRAMING PLAN

TYPICAL TRUSS ELEVATION (T1 THRU T6)

Table 1 — Member Sizes

Truss	Member	Approx. Method Member Size	Consistent Deflections Member Size
Truss T1	Top Chord	W8x31	W8x31
	Bottom Chord	W8x31	W8x31
	Diagonal	2L5x3x7/16	2L5x3x7/16
Truss T2	Top Chord	W8x31	W8x31
	Bottom Chord	2L7x4x3/8	2L7x4x3/8
	Diagonal	2L4x3x3/8	2L5x3x7/16
Truss T3	Top Chord	W8x31	W8x24
	Bottom Chord	2L6x4x3/8	2L6x4x5/16
	Diagonal	2L4x3x5/16	2L4x3x5/16
Truss T4	Top Chord	W8x31	W8x24
	Bottom Chord	W8x31	W8x31
	Diagonal	2L4x3x3/8	2L4x3x5/16
Truss T5	Top Chord	W8x31	W8x31
	Bottom Chord	2L6x4x3/8	2L7x4x3/8
	Diagonal	2L4x3x3/8	2L5x3x7/16
Truss T6	Top Chord	W8x31	W8x24
	Bottom Chord	2L7x4x3/8	2L6x4x5/16
	Diagonal	2L4x3x3/8	2L4x3x5/16

STUDIO ON STILTS

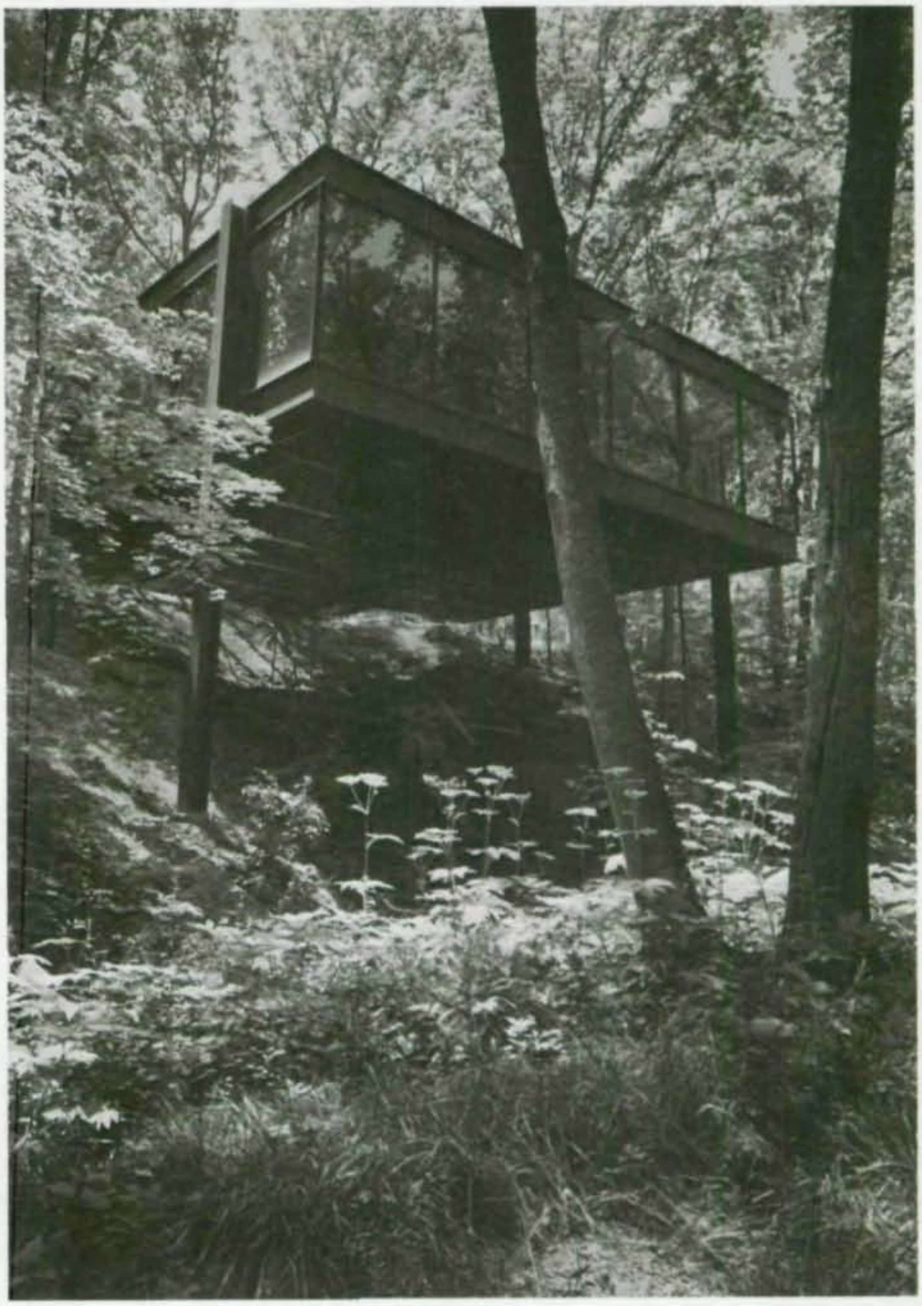
by David Haid, FAIA

A novel 42-ft x 35-ft studio, built over a ravine on a heavily wooded site near the owner's home, is designed to store and exhibit an antique car collection and provide an area for painting and weaving activities. The enclosed interior core contains a bath, mechanical equipment, and a kitchen.

The building structure, glass framing sections, and doors are of A36 steel, painted dark brown. The roof is 3/16-in. weathering steel plate insulated on the interior face. No built up roofing was used, the exposed steel serving as the weatherproof surface after being welded continuously at the roof fascia beams and joints between plates.

To reduce field labor costs and accelerate completion of construction, the entire building structure and enclosure framing were shop fabricated in 17 components. These were, the wall units, with W24X55 floor and roof fascia beams (42 ft x 13 ft and 35 ft x 13 ft); roof plates, with two W18X35 beams per plate (42 ft x 7 ft); four W12X79 columns (varying lengths); and four W21X55 floor beams (42 ft long). All shop connections were welded, using either manual shielded metal arc or automatic submerged arc processes. All exposed welds were ground smooth and flush. Despite the heavily wooded site and difficult terrain, erection was completed in one day. All field connections and joints were welded using continuous feed submerged arc equipment. Weld materials used were compatible with base metals and, in addition, contained about 2½ percent nickel. Critical connections were ultrasonically tested and all welds visually inspected and tested with high pressure water to insure complete weather seals.

Mr. Haid is President of David Haid and Associates, P.C., architects of this studio.



Architect:
David Haid and Associates, P.C.
Chicago, Ill.

Structural Engineer:
Wiesinger-Holland Ltd.
Chicago, Ill.

General Contractor:
Pepper Construction Company
Chicago, Ill.

AMERICAN INSTITUTE OF STEEL CONSTRUCTION
1221 Avenue of the Americas
New York, N. Y. 10020

Address Correction Requested

BULK RATE
U.S. POSTAGE
PAID
NEW YORK, N.Y.
Permit No. 6662

STUDIO ON STILTS (continued)

Upon completion of field welding, the floor construction (metal deck on the lower flanges of the floor beams, insulation and concrete slab) was installed. The sequence of construction put the vertical glass mullion bars in tension, thus transmitting a portion of floor load to the roof fascia beams, which for visual reasons are the same size as the floor fascia beams. This helped equalize deflections, particularly on the 42 ft span. Because of the extreme length of the columns (20 ft from the underside of the floor to base plate on the longest), additional lateral stiffness was achieved by using the entrance bridge as a diaphragm connected to the floor fascia beam and to a grade beam at the edge of the ravine.

The building is glazed with ¼-in. bronze plate glass set in the steel framing. Interior finishes are a terrazzo floor, plaster ceiling and the core of painted wood panels.