AMERICAN INSTITUTE OF STEEL CONSTRUCTION
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Chicago, IL 60611

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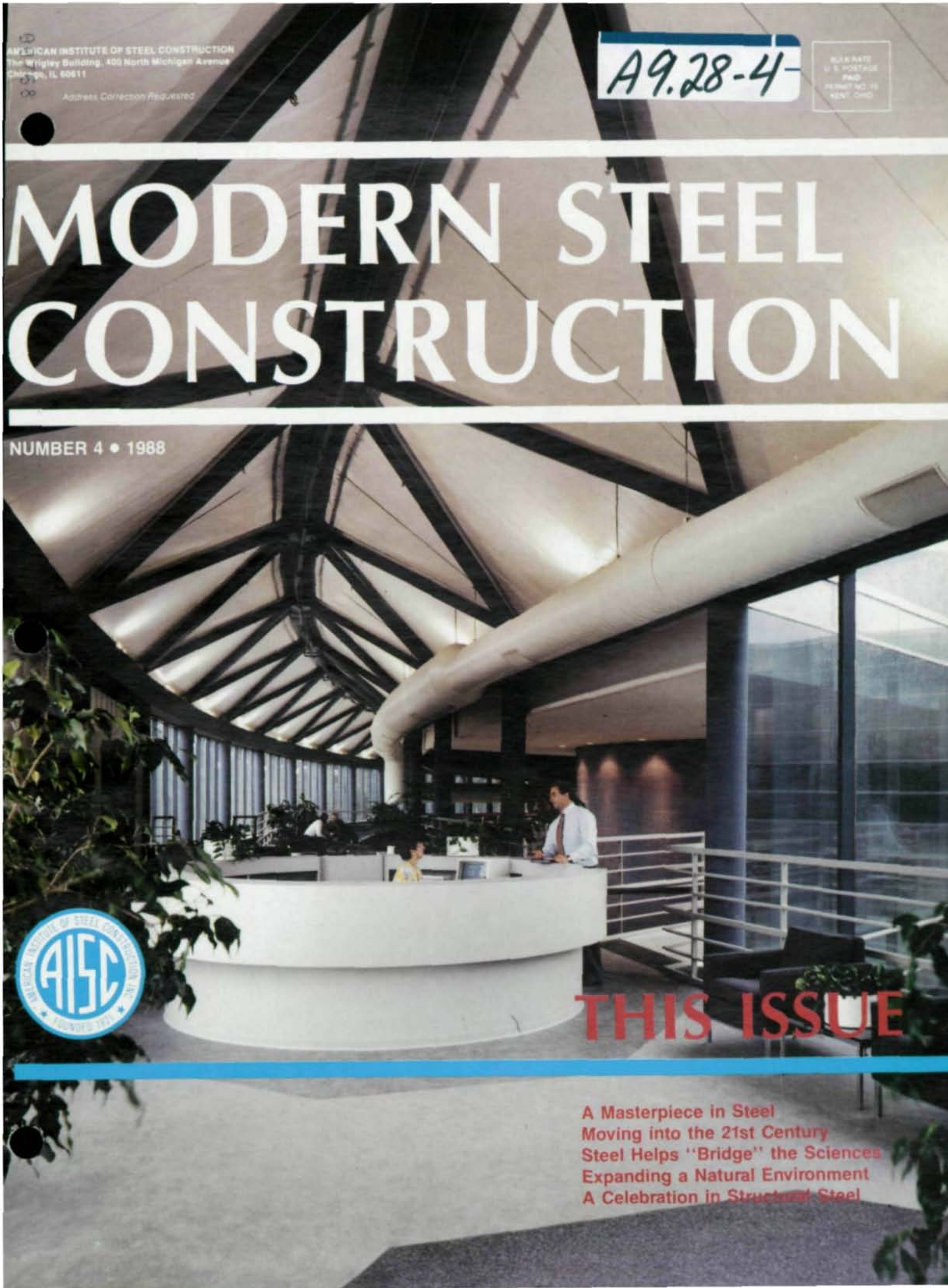
MODERN STEEL CONSTRUCTION

NUMBER 4 • 1988

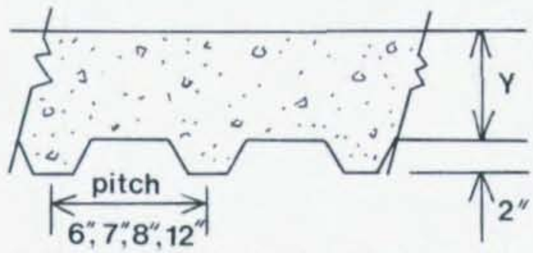


THIS ISSUE

UNITED STEEL DECK, INC.

DECK DESIGN DATA SHEET
No. 10

2" Bridge Form
(Stay In Place Steel Forms)



(Also available in 5½", 6½" + 7½" pitches)

Concrete Volume = $\frac{Y}{12}$ + Cv, $Ft.^3$ / $Ft.^2$

(2" × 12" not shown)

- - TABLE OF MAXIMUM (SINGLE) SPANS IN FEET - -

Profile	Ga	Y, Concrete Topping 5"	5.5"	6"	6.5"	7"	7.5"	8"	8.5"	9"	9.5"	10"
2" x 6" Cv = 0.0833	22	6.14	6.14	6.08	5.95	5.83	5.72	5.61	5.51	5.42	5.33	5.24
	20	6.65	6.65	6.65	6.65	6.65	6.52	6.40	6.29	6.18	6.07	5.98
	18	7.46	7.46	7.46	7.46	7.46	7.46	7.46	7.46	7.35	7.23	7.12
	16	7.95	7.95	7.95	7.95	7.95	7.95	7.95	7.95	7.86	7.77	7.68
2" x 7" Cv = 0.0714	22	5.88	5.87	5.74	5.62	5.50	5.40	5.30	5.20	5.11	5.02	4.94
	20	6.39	6.39	6.39	6.39	6.30	6.18	6.06	5.95	5.85	5.75	5.66
	18	7.22	7.22	7.22	7.22	7.22	7.22	7.22	7.22	7.13	7.01	6.90
	16	7.87	7.87	7.87	7.87	7.87	7.87	7.87	7.87	7.81	7.71	7.63
2" x 8" Cv = 0.0625	22	5.65	5.55	5.43	5.31	5.20	5.10	5.00	4.91	4.82	4.74	4.66
	20	6.15	6.15	6.15	6.09	5.97	5.85	5.74	5.64	5.54	5.44	5.35
	18	6.97	6.97	6.97	6.97	6.97	6.97	6.97	6.90	6.78	6.67	6.56
	16	7.69	7.69	7.69	7.69	7.69	7.69	7.69	7.69	7.65	7.56	7.45
2" x 12" Cv = 0.0764	22	6.03	6.03	5.91	5.78	5.66	5.55	5.45	5.35	5.26	5.17	5.09
	20	6.54	6.54	6.54	6.54	6.49	6.36	6.24	6.13	6.02	5.92	5.82
	18	7.28	7.28	7.28	7.28	7.28	7.28	7.28	7.28	7.19	7.08	6.97
	16	7.79	7.79	7.79	7.79	7.79	7.79	7.79	7.79	7.72	7.63	7.54

1. Maximum spans are based on USFHA loading
   a. construction load = 50 psf; minimum deflection load = 120 psf.
2. Allowable deflection = the least of 1/180 or 0.5". ▢
3. Allowable deck stress = 29 ksi.
4. Concrete weight taken at 145 pcf; deck and rebars estimated at 5 psf.
5. Deck Span is usually edge to edge of stringers less 2"
6. For some states the slab design depth is Y; for others the slab design depth is measured from the centroid of the form - - estimate this as Y + 1".

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MODERN STEEL CONSTRUCTION

VOLUME XXVIII • NUMBER 4
JULY-AUGUST 1988

American Institute of Steel Construction, Inc.

The Wrigley Building
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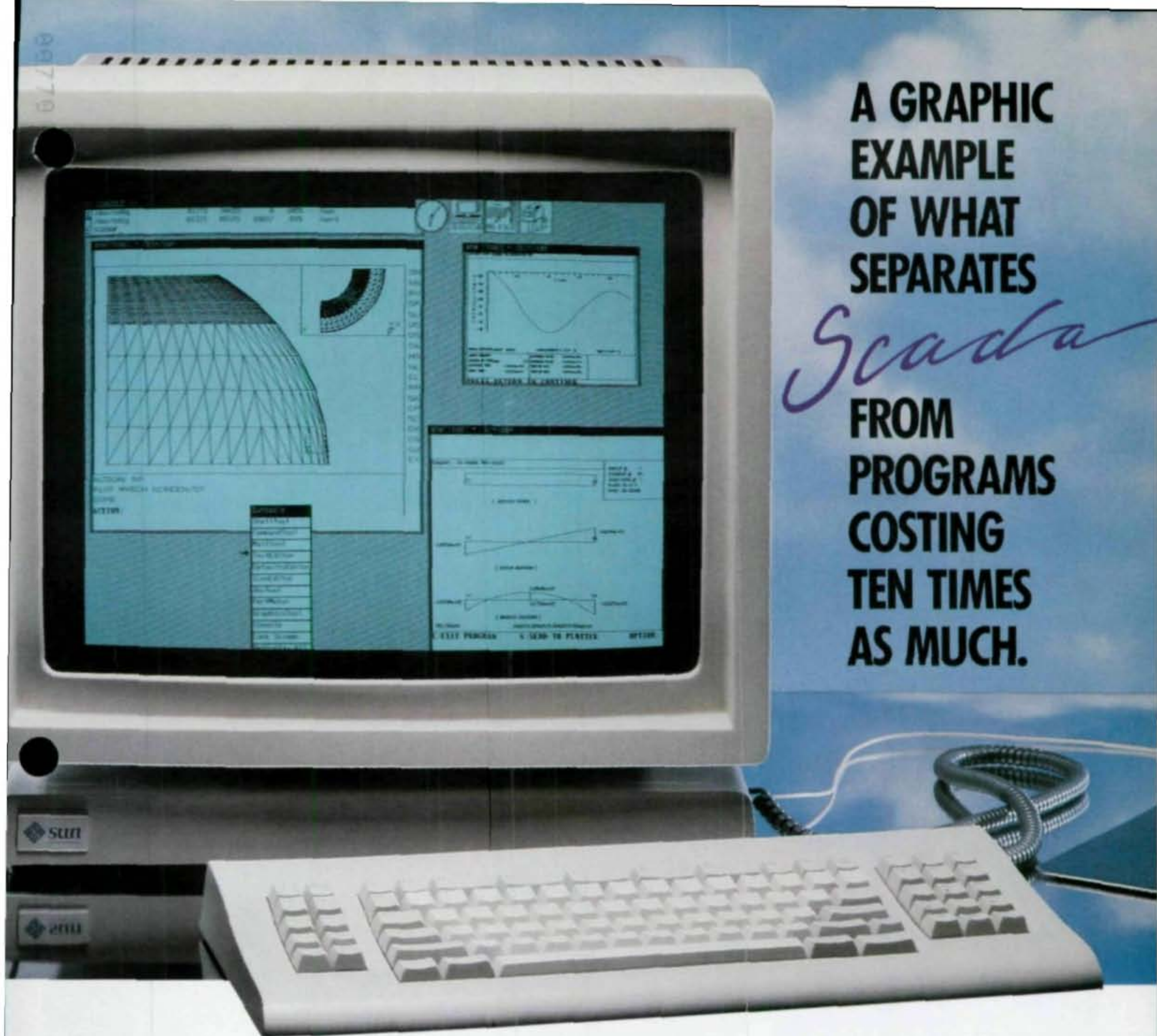
ON OUR COVER

GMF Robotics Corporation
Auburn Hills, Michigan.
Exposed steel frame of circulation spine becomes facility's design focus.

OOPs Dept.!

In Issue 3, on the Norstar Bank/Union Station project we captioned its location as Buffalo. A Freudian slip, probably because of that city's fame (for shufflin off to, Mark Russell, snow). Our apologies to Ryan-Biggs—and Albany—Editer.



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SEVENTEEN STATE STREET

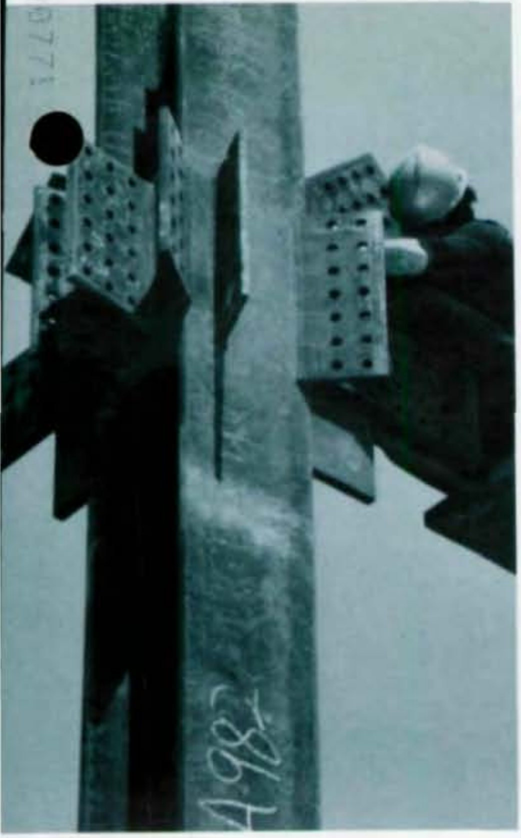
A Masterpiece in Steel

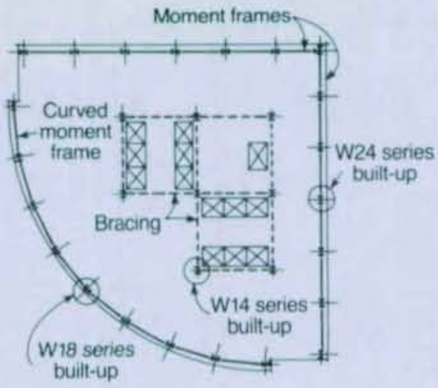
by Carlos Dobryn

At the southernmost tip of Manhattan, this enviable Seventeen State Street site presented the owner and the design team with a unique set of design challenges and opportunities. Because of its unobstructed views of historic Battery Park, New York Harbor and the Statue of Liberty beyond, the architect had to produce a building to relate to its surroundings, open up to the panorama and comply with zoning and programmatic requirements. And, accomplish all this within constraints imposed by a sensitive wind climate and a limited construction budget. The design team, including the owner, set about to conquer these obstacles and, within the context of a commitment to excellence, produced an economically feasible building that takes full advantage of its surroundings—a building that relates to the city in the background and one that can be considered a true exhibition of contemporary design in structural steel.

The Problem

Overall optimization was the code word. This tall, slender structure, 42 stories high and nearly 600 ft tall had to be designed to withstand one of the most severe wind climate areas of Manhattan. The structural solutions were limited by the logical need to provide maximum, column-free open areas and reasonable unobstructed views of the New York Harbor. Flexibility of use was important and the needs of all disciplines had to be respected with the common goal of producing a high quality building, of developing and realizing the full potential of the site and of achieving all this at a cost compatible with the economic realities of the project. The infrastructure was complicated by the need to root the building into the Manhattan rock within a crowded environment of existing old foundations.





Shape of building led to structural steel design. Required spandrel conditions on radius were more easily constructed of steel. Also, forming around corners could be done in fabrication shop.

The Solution

After a number of comparative system studies, it was apparent that structural steel had the advantage in terms of overall economy and responsiveness to the programmatic needs of the project. A higher mass density building would not have solved the need for rock anchorage and would have resulted in severe economic and programmatic penalties.

Once the material had been selected, the search for an efficient system started in earnest. The building shape—a quarter of a circle—and the off-center location of the core presented both special problems and opportunities. It was obvious the solution would have to recognize and develop the full potential ability of gravity load systems to perform at maximum efficiency under lateral load forces. Torsional forces due to building shape, stiffness center location and to some degree to mode coupling demanded a perimeter system. Tube core bracing was essential and shear lag would have to be kept to a minimum. This led to a structural system that is a combination of structural steel bundled braced core tubes and coupled perimeter frames linked by an outrigger space truss roof structure. This coupling mobilizes the wind reserve strength of every column in the building. The excellence of the solution was evidenced by the analysis results indicating high performance, low cost and responsiveness to design constraints.

Structural Features

The floor system is composite deck, composite beam construction. Interior columns are the wide-flange W14 series with corresponding built-up shapes in the lower floors. Largest columns have plates 6.5 in. thick and weighing up to 1,400 lbs./ft. Exterior columns are the wide-flange W18 series along the curved face of the W24 series along the flat faces with corresponding built-up shapes where required. Spandrel beams are W27 to W33 rolled shapes. The lateral, load-resisting system is perimeter moment-resisting frames and bundled braced core tubes coupled by a space frame hat truss. Diagonals are double-angle members.

Field bolting was the preferred connection method to reduce cost and speed up construction, with field welding kept to an absolute minimum. Two types of bolts were used on the job: 1-in. dia. A490-F for column splices, moment connections, wind bracing and girders and ⅞-in. dia. A325-N snug-tight for beam-to-beam connections. Seventeen State Street Tower is the first building in New York City to take advantage of the new provision in the Research Council's specification allowing high-strength bolts to be used in bearing without being fully tightened. The stipulation of a basic job bolt size and a different size and grade for the snug-tight A325-N bolts was an essential factor in the success of the installation as evidenced by the results of the quality control inspection program.

Foundations Rock-solid

Good rock was available, but at variable depths. A study indicated the most suitable foundation system should be a combination of steel piles and piers to rock. All major columns are on piers to rock. A number of columns were subject to uplift under ultimate wind conditions and therefore they were anchored to the rock with high-strength, post-tensioned anchors. A unique procedure was developed to avoid erection problems inherent in threading these bars through the uplift baseplate assemblies. After exposing and preparing the rock at the base of the piers, holes for anchors were drilled and anchors installed with their ends grouted into the rock. The concrete pier was poured part of the way up, at which point a simple steel falsework was installed and leveled. The column, with its baseplate assembly and anchor bars hanging down, was then set and the rest of the pier poured to its final elevation. The rock anchors were then post-tensioned to anchor the pier to the rock and finally the column baseplate was grouted. This procedure proved extremely simple and was completed in a short period of time.

Seventeen State Street, scheduled to open in early 1988, has already been judged one of the most successful buildings in downtown Manhattan. The combination of an imaginative design team and a knowledgeable owner produced a true masterpiece of contemporary engineering in structural steel. Optimization strategies and techniques in the design, fabrication and erection of the steel structure were merged and coupled with the optimization needs of the other disciplines to produce what has been called "the crystallization of 20th century structural steel engineering and architectural design." □

Architect
Emery-Roth & Sons

Structural Engineer
DiSimone Caplin & Associates

General Contractor/ Construction Manager
Lehrer McGovern Bovis, Inc.

Steel Fabricator/Erector
Steel Structures

Owner
William Kaufman, and JMB Realty, Chicago

Carlos Dobryn is project manager for DiSimone Caplin & Associates, New York, New York.

GMF ROBOTICS CORPORATION

Moving into the 21st Century—with Steel

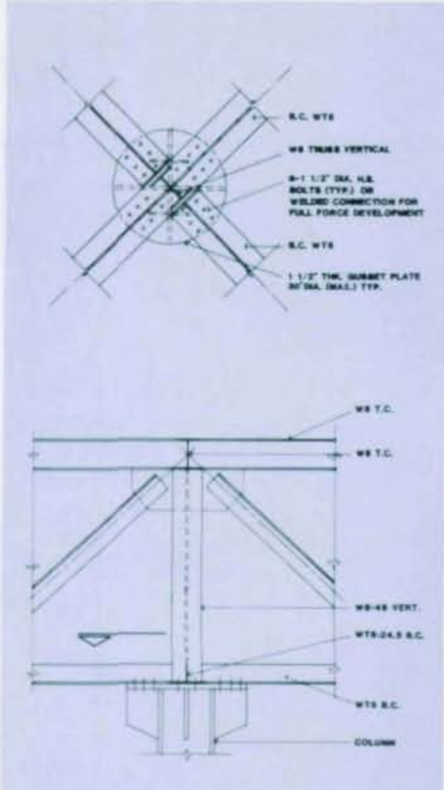
by Gerd W. Hartung

The GMF Robotics World Headquarters and Technology Development Center is one of the cornerstones of the rapidly developing, billion-dollar Oakland Technology Park in the Detroit suburb of Auburn Hills. GMF Robotics Corporation is a joint venture company created by General Motors Corporation and Fanuc LTD of Japan to research, develop, manufacture and sell advanced automation and robotics equipment. It is a progressive company with a management philosophy based on close working relationships among all employee groups. In keeping with this philosophy, a single, multi-area complex was needed to integrate the showplace corporate headquarters with state-of-the-art research and development areas, including manufacturing facilities and a training center.

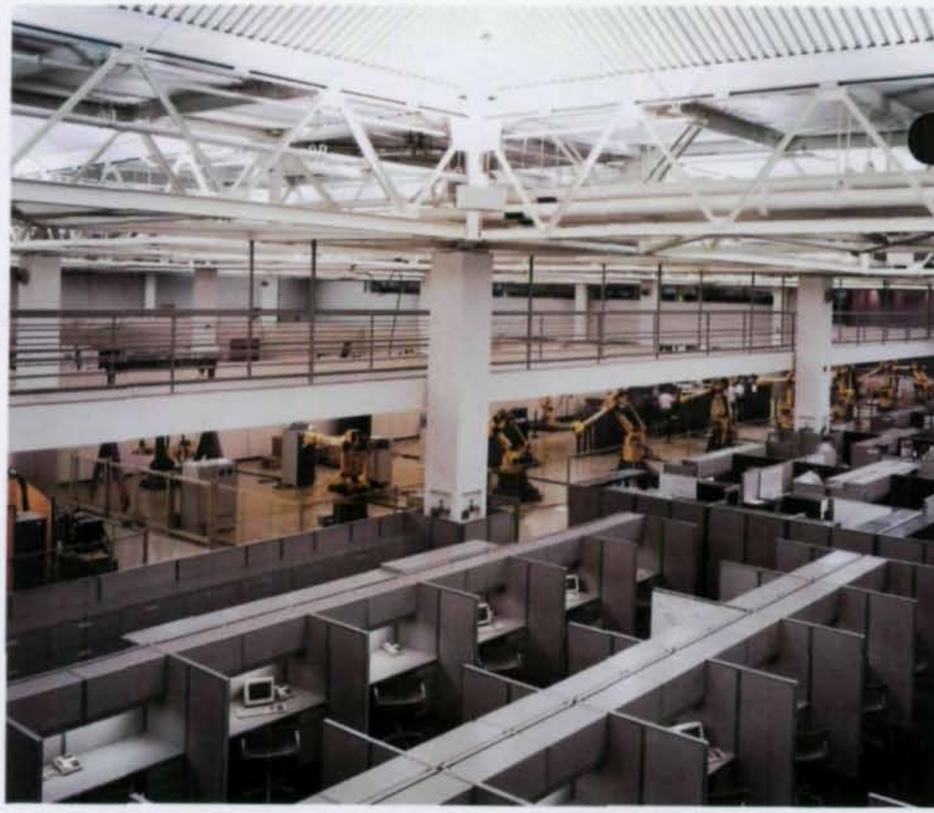
Design Parameters

The project architect/engineer had the responsibility to develop and deliver a design incorporating a variety of concepts which would combine to achieve a physical manifestation of GMF Robotics' distinctive corporate philosophy. These concepts required the building's ultimate image be established with the first phase, although expansion was anticipated regu-

Design and planning elements unite to create natural blending of building and environment.



Elevation/plan details of column pod



Catwalk, traversing Technology Development Center, overlooks temporary engineering stations (at r.) and finished product area (l.).

larly. Offices and technical development areas had to be consistent in appearance, while flexibility and multiple use of spaces was essential for effective functioning.

Making use of the extraordinary natural vistas within the heavily wooded site was very important. Maximum retention of trees and respect for the adjacent land was essential. GMF insisted the new facility express the best image of their products, although it was not to be extravagant or ostentatious. The desire was to reflect an image of a cost-conscious, quality company.

Successful integration of these requirements resulted in an approximately 320,000-sq. ft complex of four disparate building types: Technology Development Center, office and administration areas, spine and ancillary structures. The former three have been completed, using approximately 1,820 tons of A572-50 and A36 steel for support. The ancillary structures are designed and will be constructed in subsequent phases.

Structured Amenities

The Technology Development Center is the heart of the complex. The framework is exposed structural steel, except for fire-protected columns. Built to a 50-ft × 50-ft column grid, it offers few lateral constraints. To provide the maximum flexibility desired by the owner, vertical braces were not used. Instead, lateral stability is pro-

1989 NATIONAL STEEL CONSTRUCTION CONFERENCE

Opryland Hotel

Nashville, Tennessee

June 21-24, 1989

Trusses are connected into column pods in structural steel frame of TDC.

vided by moment frames. A suspended catwalk extends throughout the Center, providing visitors and staff with a bird's-eye view of both work in progress and finished products.

The building perimeter has extensive glazing along the wooded "nature" side as well as at the main circulation spine. Employee morale stays at a high level in an open, well-lighted work environment. Conventional windows would lose valuable natural light to teams working away from the perimeter of this 250-ft × 450-ft area. Therefore, extensive skylighting ensures a liberal amount of lighting. Each bay has a 12-ft × 12-ft skylight, positioned on a 35-ft × 35-ft exposed steel pyramidal base. The bases' sloped surfaces have a white coating which reflects and diffuses available light. During periods of low natural light, artificial up-lighting is reflected from these bases. The result is a shadowless, inviting, open and well-lighted floor space. Neither ducts nor pipes penetrate these pyramids.

Column Pod

Integrating the structural requirements and the desired mechanical and electrical paths resulted in a two-way moment frame to support gravity and wind loads. The normal approach of columns and support trusses in both directions parallel to the column axes was investigated. This created some interference concerns, which

Prototype robots in line for testing in Technology Development Center

Steel members frame distinctive curved circulation spine.

were compounded by the additional clearance requirements for the skylight support members. Interference problems for mechanical and electrical system distribution could have been minimized by introducing columns at each corner of the skylight. This idea was not pursued because it severely restricted the required flexibility of the floor space.

Instead, a column "pod" was developed. Outrigger trusses, short and deep, extend 15 ft from the columns. They are aligned at 45° in the horizontal plane from the column axes. These skewed trusses terminate at the pyramid corners. The result is a column with an effective 15-ft × 15-ft × 5-ft rigid box top. The skylight supports are framed at these points into the skewed trusses. Extensive use of in-house, computer-aided analysis and design checked design and clearance requirements. Ducts up to 39 in. dia. can now be accommodated for either a straight run or a 90° turn within the pod area.

Economical Solutions

The office and administration areas feature standard nuts-and-bolts steel construction—a composite deck and composite beams. Columns are spaced on an economical 25-ft × 25-ft grid to complement the Technology Center's 50-ft pattern. Cellular deck provides for the distribution of electrical power and services. Duct shafts are located within circular masonry cores at the extremities and shear walls within these cores provide lateral stability. Locating these cores outside the office areas ensures utmost flexibility.

The 800-ft long curved spine is the design focus of this project. Its sweep of diagonally intersecting frames hugs the western edge of the Technology Development Center. Exposed steel tubes were used in the construction of the frames. Four diagonals intersect at the ridge in the center of the spine. Continuity was required. Simplicity of design, both architectural and structural, led to an economical solution: a hub, hexagonal in plan, which permitted the tubes to fit at right angles. Welding finished the assembly, to create a frame with structural continuity and aesthetic design.

Future development of the ancillary structures—cafeteria, technical training center, boardroom and marketing presentation room—will use the spine as a springboard. The transitional areas between the rectangular technical spaces and the curved linear spine are planned green spaces—informal meeting and brainstorming areas.

Steel the Logical Choice

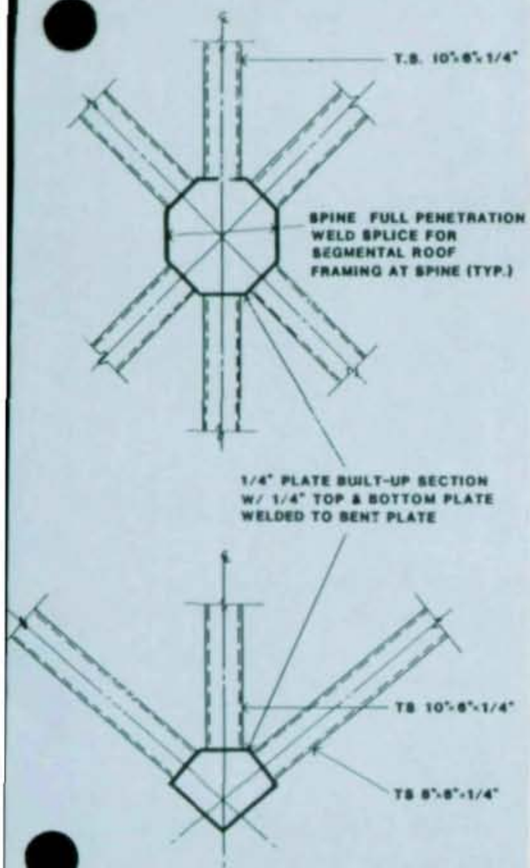
Steel was, by far, the most logical choice for the GMF Robotics headquarters. The Technology Development Center, with its high bay and long span, has a structural support system relatively light in weight for the given spans.

The Office and Administration areas benefitted from using extensive composite action. The owner received a flexible structure of open-ended design with in-slab electrical distribution. Exposed steel framing became part of the architectural declaration of the spine—it complements the gentle curvature without overpowering it.

Steel construction permitted the contractor to make use of extensive, on-site fabrication prior to erection of components. The Technology Development Center columns received their 15-ft pods on a jig rigged at the site. They were then moved to their proper location. Similarly, the 35-ft skylight bases were assembled before erection.

This pre-assembly resulted in economies because of the proper fit and easy access for welding and quality control. And provisions for future expansion were included with only modest cost premiums. The architect/engineer successfully delivered the GMF Headquarters complex by addressing and incorporating each of the stated design goals.

The use of structural steel greatly facilitated the task! □



Spine roof plan details

Architect/Engineer
Harley Ellington
Pierce Yee Associates, Inc.

General Contractor
Barton-Malow Company

Steel Fabricator
Carter Steel & Fabricating Co.

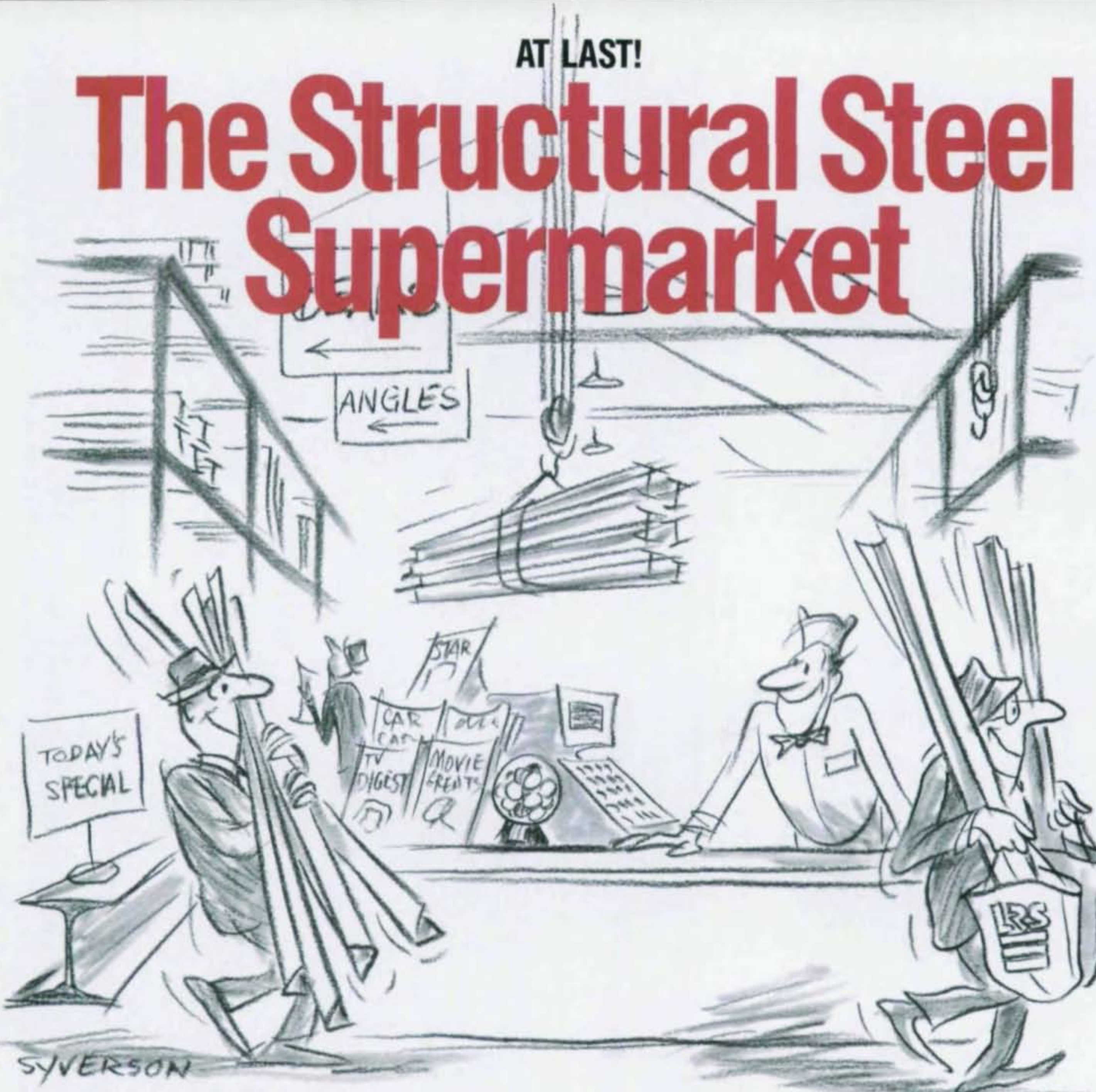
Owner
GMF Robotics Corporation

Gerd W. Hartung, P.E., is director of structural and civil engineering for Harley Ellington Pierce Yee Associates, Inc., Southfield, Michigan.

Pijush K. Chowdhury, Ph.D., P.E., was assistant director of structural engineering and lead structural engineer for the GMF project. An untimely death prevented him from seeing the completed fruits of his labor.

AT LAST!

The Structural Steel Supermarket



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BECKMAN INSTITUTE

Steel Helps Bridge the Sciences

by John P. McCarthy

Flexibility in structural design is a must for a building like the University of Illinois Beckman Institute for Advanced Science and Technology. Maintaining the Institute's planned state-of-the-art capacity challenged architects and engineers. Providing for future, unplanned needs presented even greater challenges. A steel structural system provides the perfect material to meet unexpected needs created by future scientific and technological advancements.

When construction is finished on the Institute, scientists and researchers take over from architects and engineers. Major advances are expected in computing, medical technology, neuropsychology, molecular biology and other fields, including factory automation, robotics and improved air-traffic safety. The Institute has two centers—a Center for Biology, Behavior and Cognition and a Center for Materials Science, Computers and Computation. Researchers and scientists will attempt to bridge the physical and biological sciences on a scale comparable to no other interdisciplinary research institute in the nation. Funded primarily through Arnold O. and Mabel M. Beckman's $40-million contribution, the 309,000-sq. ft., $50-million complex represents the realization of their lifelong dream.

Model photo, south elevation of Beckman Institute

Site with a History

The Institute is being constructed where U. of I.'s first building once stood. When the University opened in 1868, one building housed all offices, departments and dormitory rooms. This main building was almost demolished by an 1880 windstorm. By 1881, the site was a vacant lot and the bulk of the university had relocated to the south end of the campus. After 10 years, this lot became Illinois Field, and U. of I.'s baseball team opened its spring season on the field that would serve their athletic department for almost 95 years.

One Building, Three Structures

Structurally, the Institute is three separate buildings: a main block of labs and offices, a column-free auditorium and a 125-ft tall campanile tower. Its floor system is 3¼-in. lightweight concrete topping on 3-in. metal deck to provide a two-hour fire rating—all supported by composite steel beams and girders.

The main block is a three-story laboratory and a five-story office tied together by double-angled bracing members expressed in the connecting atrium. Common areas—conference facilities, administrative areas and public spaces—are located in the atrium, making it the ideal place for students, faculty and visitors to meet.

Steel construction of atrium roof members

A 200-seat auditorium at the west end of the first floor required a column-free space below three floors of offices. Several schemes were investigated, including hanging the second floor from above or providing a large transfer girder at the second floor to support the floors above. The framing system used was to free-span all the floors in the auditorium area and use rigid, moment-resisting frames to resist lateral loads. To reduce the ponding effect, beams were cambered. Long spans were investigated for vibrations, which were of concern, especially in the labs.

The third structure, a 125-ft tall campanile tower, houses main conference rooms. The roof is crowned with a 19-ft square glazed cap lighted from within at night to produce a beacon effect. Beneath the lighted cap is a two-story, glass-walled conference room. Located at the north end of the campus axis, the tower is the most dramatic architectural element of the project.

The tower and connecting bridge are separated from the main block by an expansion joint. The base of the tower, which spans the ceremonial entrance, consists of four massive concrete columns extending to the second-floor level. A braced steel frame extends above this. Like most of the building, the bracing is exposed and will provide an exciting architectural element, particularly for the large conference room at the top floor.

Structural Challenges

The connecting bridge of the tower provided structural challenges because it acts as a large sail to produce considerable torsion in the tube-shaped tower. Also, the roof of the bridge could not be picked up by the tower because of elevation differences and curtainwall requirements. This meant a post had to be added at the fourth floor, through the fifth floor, to provide lateral restraint and support the bridge roof.

The atrium roof also presented challenges. In the atrium, which divides laboratories from offices and provides natural lighting through three 34-ft by 60-ft skylights, the required "clean" look was provided by continuous, exposed steel tubing. Care was needed in design, fabrication and detailing of the skylight supports because possible damage from snow drifting was of considerable concern. The differential building elevations and the "bathtub" effect produced by the skylight configuration required a high-design snow load. A snow accumulation test was performed which indicated the

Photo of model from northeast

atrium roof could experience snow drifts of over 7½ ft. Long clear spans, high loads and rigid-frame design called for large, 20-in. by 8-in. steel tube sections. Field-welded connections provide a clean looking structural system.

The curved roof covering the lab presented additional problems to structural engineers. The original design concept called for girders to be bent to a radius of about 80 ft. But the steel fabricator felt it would be more cost-effective to provide straight beam segments and vary the beam and joist connection heights to provide the required curvature at the metal roof deck. Unfortunately, this change complicated the framing at the intersection of the curved portion and the dormers. The resulting connection bracket was skewed and twisted to meet with beams which were bent to approximate the mathematical curvature of the roof's intersection with the plane of the dormer roof.

Unique Complications

The Institute's north half is devoted to sophisticated research laboratories which require large quantities of piped gases and liquids. Conditioned air is also a constant requirement. The entire mechanical distribution system had to fit into the ceiling and additional space had to be made available for future growth. These me-

AFE FROM EARTHQUAKES MPANIES THE SHAKES.

That was essential because the building, which was constructed for Evans & Sutherland Computer Corporation, is located within a mile of the Wasatch Fault in Salt Lake City. What's more, Evans & Sutherland is a leading designer of special-purpose digital computers, software systems and display devices — products extremely vulnerable to damage from seismic tremors.

To plan for maximum protection, Vulcraft was asked to join with the architects and engineers at the design stage of the project. Already, they'd decided to use a "base isolation" system, the most advanced buffering method available. But using our steel joists and joist girders was also an important decision. The joists and joist girders are much lighter in weight than wide flange beams, so the entire building required less steel, lighter columns and less foundation. And this not only lightened the load for the base isolators, it saved appreciably on building costs.

Throughout construction, Vulcraft remained constantly involved, tailoring our delivery of materials to the exact erection schedule and meeting deadlines without fail. What's more, our joists and joist girders helped the steel erectors meet their deadlines. That's because our products are fast and easy to erect — a fact that saves time and money on virtually any job where they're used.

The design team chose a "base isolation" system to accommodate seismic loads. They also specified Vulcraft steel joists and joist girders for the building.

So whether you need Vulcraft's help to protect your building from earthquakes or you want to stay out of the hole when it comes to construction costs, contact any of the plants listed below. Or see Sweet's 05100/VUL.

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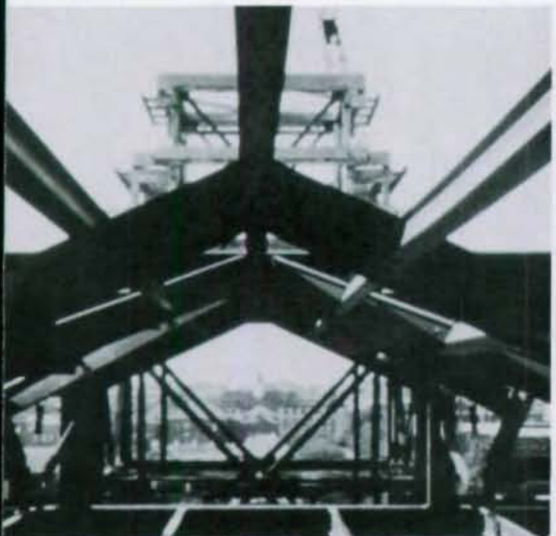

Steel roof framing, looking toward campanile tower.

Closeup of campanile tower

chanical constraints dictated a 16-ft floor-to-floor dimension.

The attic floor, which provides a fire rating above occupied spaces, was set 15 ft above the finished floor. This was adequate for the office side, but did not work for the lab section. The solution was to hold the lab's center bay to 16 ft and slope the outer bays 1 ft to match parapet details around the building. Because the building perimeter steps out 2 ft every two bays, the exterior spandrel girder was picked up by a relatively shallow beam section. The spandrel girder which was dropped 1 ft, intersected below the beam which was not dropped. A 3-ft long piece of wide flange was welded to the shallow beam to create a deep web section for the connection.

Computers will be used extensively at the Institute, so most of the building has a raised access floor. To insure handicap accessibility, the floor slab was dropped 6 in. to provide a completely level surface. The office floor was relatively easy to design because the entire floor was depressed, with only a small account of concrete fill required. The lab had only limited portions of floor area which needed access floor. Typically, an entire bay was dropped with the concrete fill used at corridors. Complications arose when the depressed floor extended beyond the girder, cutting into the beam on the other side. A large cope was required with a web extension welded on below the flange to provide sufficient shear area. Stiffener plates at the top of the cope provided the flexural capacity at the reduced section. AISC's publication, *Engineering for Steel Construction*, was used extensively in designing these and other unusual connections.

Also, on the lab side of the building, depressions were required under environmental research rooms. Steel framing made these isolated depressions extremely easy to detail, accommodating all conditions.

The permanence of the Beckman Institute as a world-class research facility will be a tribute to team members' hard work and careful planning. Fast-track scheduling and budgetary constraints added difficulty to an already complex task. The use of structural steel permitted simple and economical solutions to many of these difficulties.

Not only did the project come within budget, it also is several months ahead of schedule. □

Architect/Engineer
Smith, Hinchman & Grylls Associates, Inc.
Detroit, Michigan

Construction Manager
Turner Construction
Chicago, Illinois

Steel Fabricator
Zalk Josephs Fabricators, Inc.
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University of Illinois
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John P. McCarthy, P.E., is a structural engineer at Smith, Hinchman & Grylls Associates, Inc., Detroit-based architects/engineers/planners.

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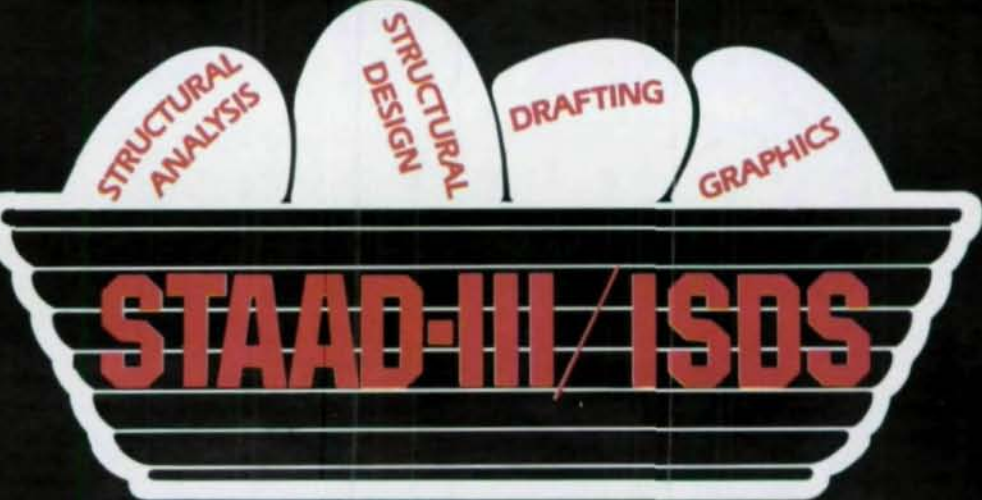

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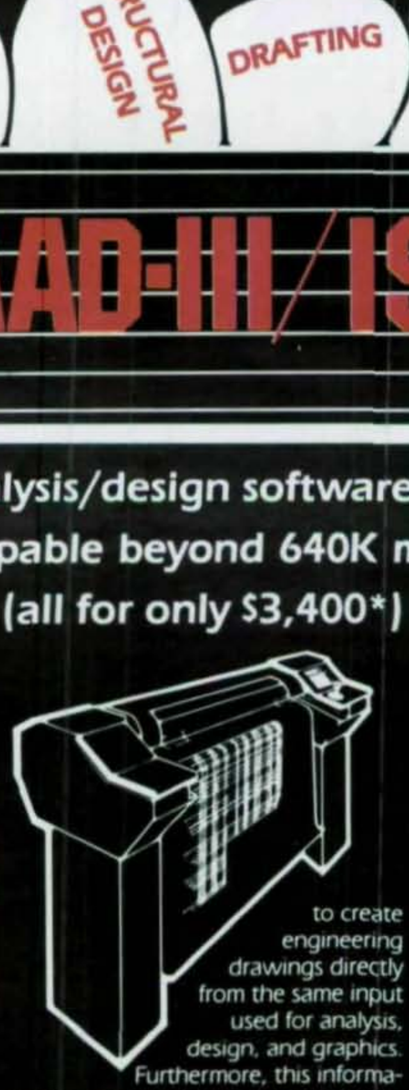

to create engineering drawings directly from the same input used for analysis, design, and graphics. Furthermore, this information is held in a central database maintained by ISDS. User interaction is minimized, improving both efficiency and data integrity.

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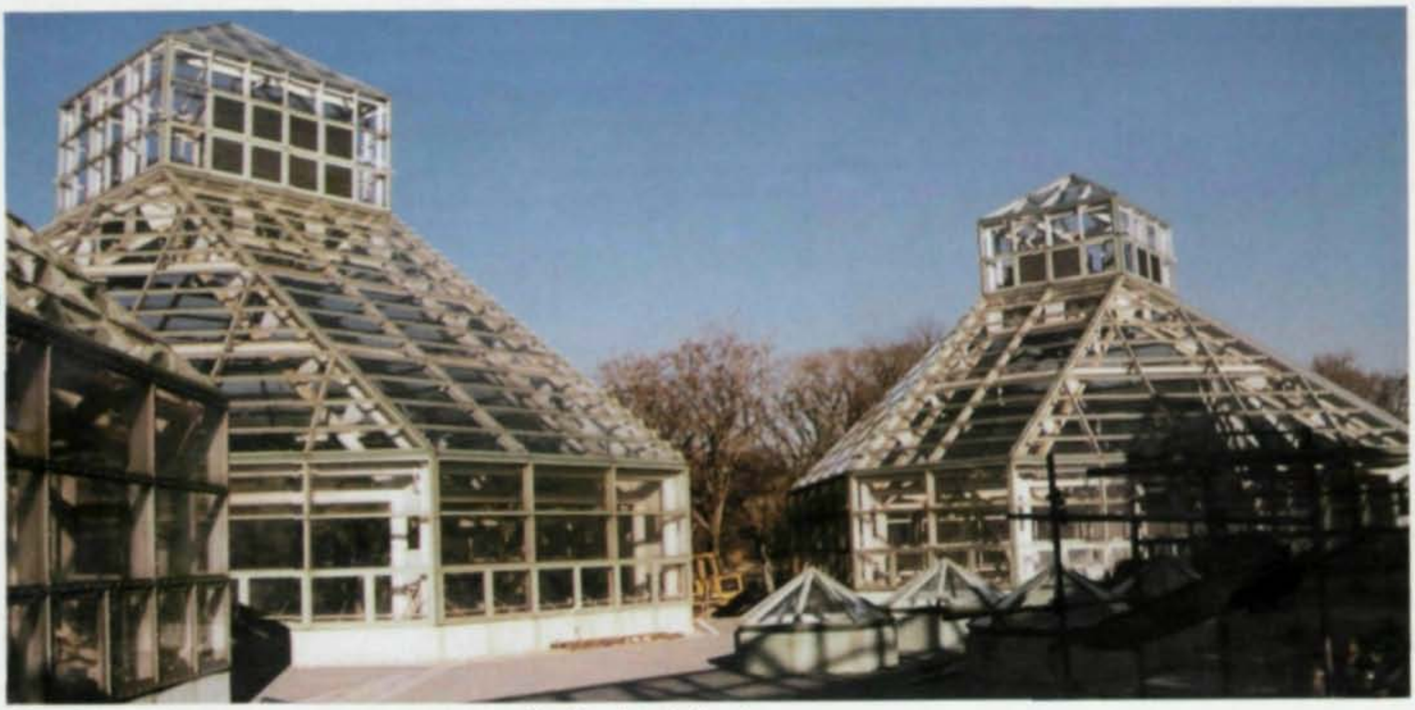
Botanic Garden Conservatory

Expanding a Natural Environment

by Joseph D. Goldreich



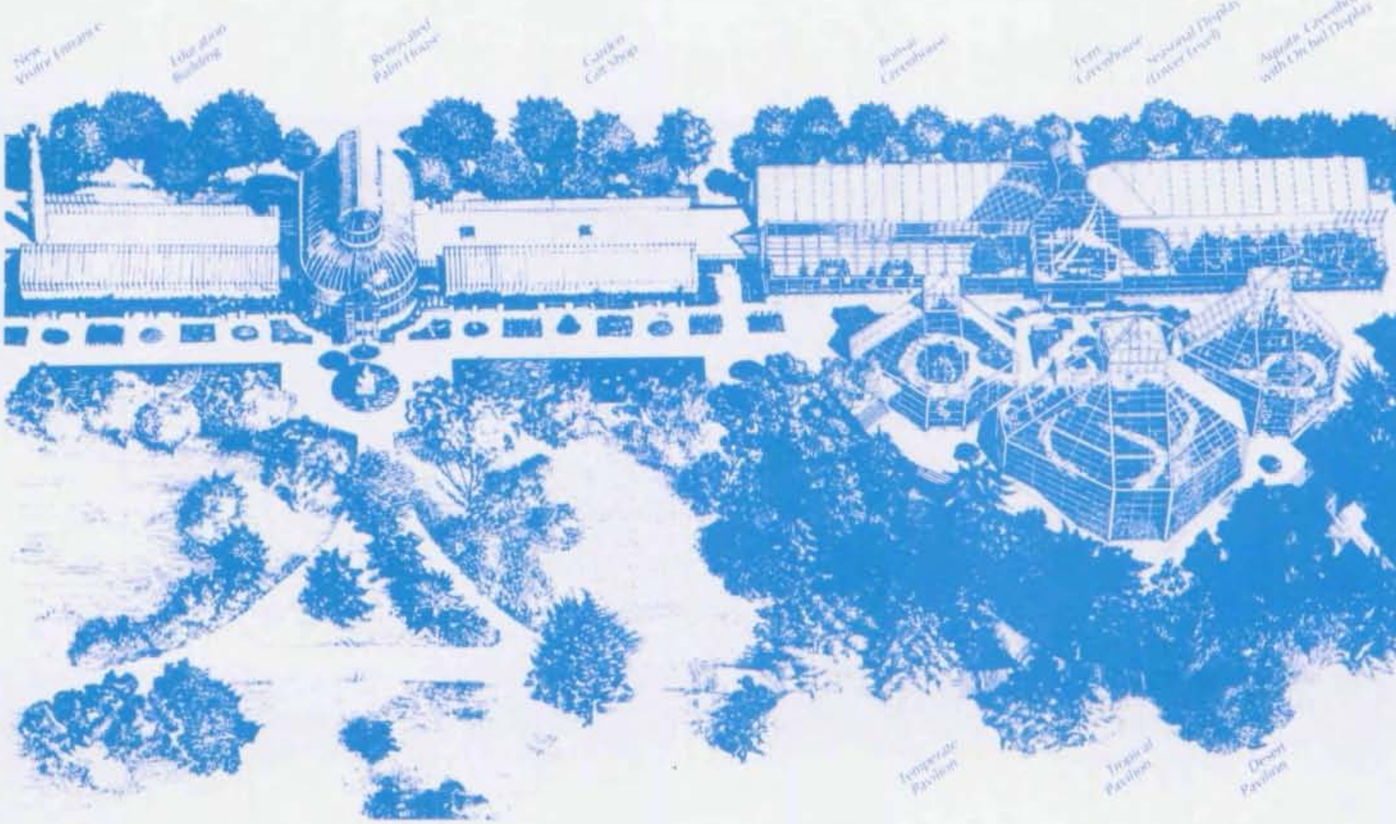
Overall view of three new conservatory structures at Brooklyn Botanic Garden

When the architect was awarded the commission to plan a conservatory renovation and addition to the Brooklyn Botanic Garden by the New York City Department of General Services and the officers of the Garden, no one imagined the interplay of complexities that would result.

The design team was asked to create a facility to capture the imagination of visitors, help expand their knowledge of the natural environment and increase the number of visitors from 600,000 to one million per year. To accomplish these aesthetic, educational and practical goals, the architect wanted the structures to have a very light appearance—and the feel of greenhouses. At the same time, they obviously had to meet the standards for public assembly buildings. As the plan developed, all this was to be done in the design of two new rectangular greenhouse-type buildings joined at an octagonal fern house, and three detached octagonal greenhouse pavilions.

Following a master plan devised by Frederick Law Olmsted in the early part of this century, and maintaining compatibility with the existing Beaux Art architecture of the Italianate administration building designed by McKim, Mead & White in 1918, the architect designed a 90,000-sq. ft expansion to the Botanic Garden that respects the landscape, the architecture and the delicate openness of the Garden itself.

To accomplish this, the building structures are sited along the edge of the garden, with exhibit and circulation space at or below grade level of the area. Reinforced concrete was used to make the transitions possible. The main plaza itself, the central space between the exhibit structures, is the top surface of a waffle slab, the underside of which is exposed and used for dramatic cave effect.



Rendering of project

Project Dominated by Superstructure

The project is dominated by the new steel and glass superstructure. But the concrete construction in place far exceeds what would normally be foundation and slab work for a steel building. Concrete in massive piers, retaining walls, aquatic growth pools, waterfalls and grand stairs is the backdrop for fern gardens, bonzai collections, water plantlife, as well as the base of separate pavilions for tropic, arid and temperate plant species. Changes in exterior grade across the site required retaining walls of continually varying heights. The octagonal shape of the steel-frame superstructure created a never ending series of crooks and corners in the concrete foundation work. Together, the architect Brody and the structural engineer decided to use tubular structural steel in an intricate series of exterior planed surfaces to achieve material dimensions of least visible impact and intrusion. To make this work, these considerations needed to be addressed:

1. Achieve use of smallest (i.e. less visible) structural shapes by using continuous framing through rigid joints. The principles from industrial applications that are seldom, if ever, used in commercial/institutional design were adapted. Vierendeel space trusses of job-dedicated configuration were developed and designed.

Typical greenhouse roof framing

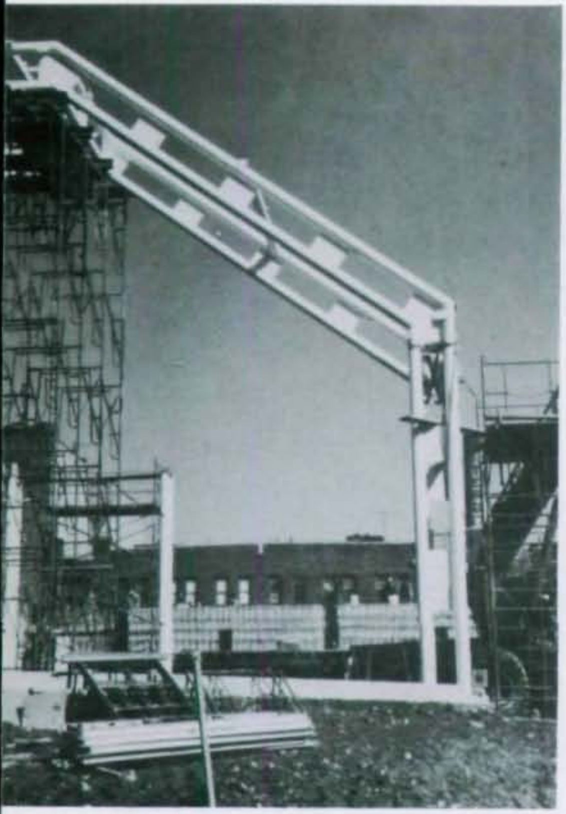
Tropical house showing six-tube connection

Tropical house interior

2. Provide for a corrosion-proof structure in a necessarily humid environment with total welding of joints. This afforded hermetical sealing of the pipe structure and full continuity of the framing. It was necessary to develop a methodology of fabrication permitting interpretation of X-ray plates which would reveal whether or not the welds were acceptable for structural integrity.
3. Accommodate a sloping site which dictated the base elevation of each vertical leg of each space truss was different. The team had to fix the dimensions so everything would fit and to devise an analysis method which could be accommodated by the capacity of their computers.
4. Account for eight directions of wind on each of the three individual buildings acting through pipe joints that in some cases had as many as eight members intersecting at a common point, for the transmission of axial and bending stresses. 4,608 individual coordinates in space were analyzed three-dimensionally for stress and deflection.

Tubular Steel System

The tubular steel structural system is composed of members that are in truss and individual assembly using a $10\frac{3}{4}$-in. dia. and a $8\frac{5}{8}$-in. dia. ASTM A500B, 46-ksi yield point structural round tubing and



Trusses erected (r).

Prefabricated space trusses as they arrive on site.

ASTM A36, 36-ksi yield point structural steel plates. A single diameter tubing was used in a single building to facilitate fabrication and erection. All welding, shop and field, was done with E70XX series electrodes.

The frames developed for this project are two pairs of bent "space trusses" rising from individual base plates to intersect at right angles at their crowns, pass through each other and descend back to base plates on the opposite end. Each bent space truss is a double line of parallel tubes two feet apart and connected by 2-ft long by ½-in. thick steel plates to produce Vierendeel trusses. The plates penetrated windows in the tubing so they could be welded to both the far and near shells of the tubes they connected. All pipe joints are a through-plate type designed to transmit axial and bending stresses as continuity of the framing through all support points, joints and connections was the prime design concept. This concept added to the complexity of the three-dimensional lateral force analysis of the structures, but was necessary to minimize deflection of the tubes on the lengths they span, a critical parameter considering the planned glass facade.

For the humid conditions essential to simulate the natural environment of the plant and tree exhibits, material choice and

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1989
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ABSTRACT FOR PROPOSED 1989 PAPER/PRESENTATION
(See Reverse Side for Abstract Guidelines, Preparation of Final Papers, etc.)

Return this form **before September 30, 1988** to:

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Call for Papers

The 1989 National Steel Construction Conference will be held at the Opryland Hotel, Nashville, Tennessee, June 21-24, 1989. Participants will include structural engineers, fabricators, erectors, educators, and researchers. Potential authors are requested to submit abstracts of papers on design, fabrication, and erection of steel structures for buildings and bridges.

Guidelines for Abstract Proposals

* Abstracts for Papers to be considered for presentation at the Conference must be submitted to AISC **before September 30, 1988**.
* Abstracts should be approximately 250 words in length, and may be typed directly on the lower portion of the reverse side of this application, or submitted on a separate sheet of 8½ x 11" white paper attached to this submission form.
* Authors will be informed of the Organizing Committee's decisions by November 15, 1988. Successful authors must submit their final manuscripts for publication in the official 1989 Conference Proceedings by April 1, 1989.
* Registration fees for the Conference will be waived for the Primary Author presenting a paper at the Conference.

Preparation of Final Paper

Final manuscripts for publication in the official 1989 Conference Proceedings are expected to be approximately 20 pages in length, copy (including photographs) must be camera-ready. Complete instructions for preparation of final manuscripts will be forwarded to authors upon acceptance of Abstract Proposals.

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specification provisions for adequate protection and surface finishing were an important consideration. The structural requirement for welding afforded the by-product of hermetically sealing the pipes against internal corrosion. For best possible protection prior to fabrication, rust-inhibitive coatings were specified for application to the interior wall surface as soon as practicable after the pipe was produced. The surfaces of all welds at joints and window fill-ins were ground smooth and faired-in to the pipe diameters, leaving no edges, ridges or pockets in which moisture could collect and rust begin. Three coats of shop paint were applied and field touch-up followed the same procedure. Grinding down all welds also served the architect's aesthetic purpose of inobtrusiveness, in that connections as such, do not appear to occur—pipe seems to just flare into pipe.

Structural Analysis Critical

For the structural analysis of the structure and its components it was necessary to assign wind forces, windward and suction on the sloping and yawed surfaces of the building structures. The forces are related to the angle of attack between the wind direction and the plane surface impacted. If there are eight vertical and sloping faces in each of three buildings with differing base elevations for every frame, the magnitude of the structural analysis becomes evident.

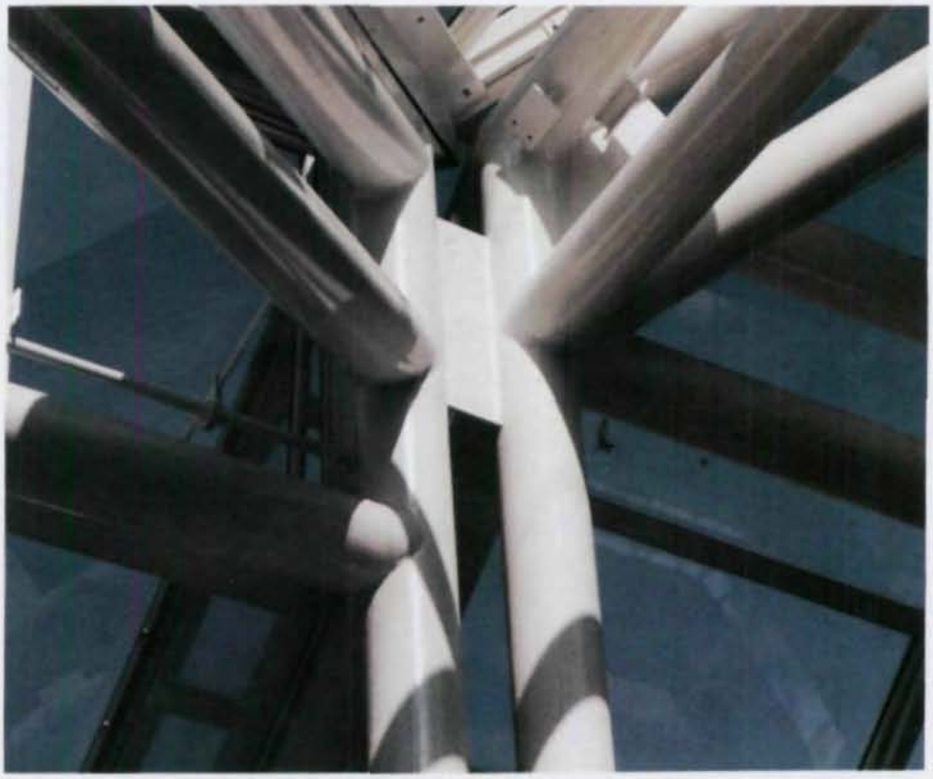
Thus, in the analysis and design of the three octagonal conservatory pavilions, eight directions of wind on each of the three individual buildings were considered. Each building had 64 analysis coordinates (nodes) at the knees (eaves) and 128 analysis coordinates (nodes) for a transverse frame. The total number of individual coordinates in spaces analyzed three-dimensionally for stress and deflection were $3 \times 8 \times (64 + 128)$ equalling 4,608. The most critical condition was chosen for final design. The hardware used for this structural analysis was a Xerox 820II PC with 8-in. floppy disks. Software was the SAP 81 program as developed by Structural Analysis Programs. The software was applied in successive stages to remain within the capacity of this system.

Magnitude of Computer Time

These schedules of loading and analysis conditions show the magnitude of the work required to produce the totally exposed structure of the Garden buildings, which becomes light and almost unseen in the context of the facility—just as the architects wished it.

Wind-lbs. per sq. ft	Windward Pressure	Leeward Suction
Perpendicular to surface	20	8
Purlins supporting glass mullions	27	18
Sidewall elements supporting glass mullions	30	20
Dead Load-lbs. per sq. ft		
Glass skin and mullions	10.5	
Fixtures & mech. & elec.	5	
Roof Live Load-lbs. per sq. ft	15	

The controlling design parameter, other than stress, was a maximum, single member deflection under wind and live load not to exceed ¾ in. under the most critical combinations of load:



Fern house joint

Total dead load + wind load
Total dead load + live load
Total dead load + 0.5 live load + 0.75 wind load.

Of the three individual pavilions, the largest—the Tropical House—required 90 minutes of computer time to run just one case of three dimensional wind stresses and deflections, with an additional 40 minutes for print-out. Compare this to another structural design project, a six-story, 36,000-sq. ft office building which required only 20 minutes of in-house computer time. The joint design was produced and indicated on the schematic layouts on the drawings. There are no eccentricities in the design, and the centerlines of all connected pipe members passed through a single working point.

The joints were shop fabricated as one weldment as designed. Trussed columns and frames arrived at the jobsite with sleeve extensions projecting from the pipes. The only field welding of the space trusses was a full penetration butt weld around the pipe circumference, which attaches it continuously to the adjacent pipe extension which in turn serves as a directional template and weld backup piece. Fillet welds done in the field were used for pipe purlin connections in the pavilions and greenhouses. The steel was erected using falsework towers, with care taken to line up every piece with a tolerance of only ⅛ in. allowable.

In fabrication of the space trusses, the pieces had to be milled, and each joint received complete penetration welding. Shop welding procedures were reviewed carefully by the fabricators, our office staff and the welding inspection agency. The relative merits of the inner shield method and MIG procedure were weighed and checked with officials of the American Welding Society. The MIG procedure was used, with special attention to the individual welder's expertise in controlling the amperage to prevent a short circuiting transfer. Painstaking care by the fabricator provided miters and chamfers on the pipe walls and joint stiffener plates so that completed welding could be inspected by X-ray and yield film views interpreted suitably.

Since the AWS *Weld Manual D1.1* does not provide guidelines for welding the modified type of Y, K and T joints resulting from the structural system of this project, it was necessary to develop a methodology of fabrication and inspection interpretation to insure weld integrity. In effect, it was close to impossible, on first attempt, to inspect pipe joints radiographically. But many full-size samples were produced through the cooperation of the fabricators, the inspection agency and the engineer to reach the "land" dimension of the joint plates into the pipe wall. This was done to obtain an observable full-penetration weld. All inspections of welds in the shop were predicated on 100% coverage for knee joints with X-ray and dye-penetrant and magnetic particle for fillet welds. In the field, at the slip-joints, after 100% X-ray inspection of the first 10 full-penetration welds, and with no rejections, the inspection rate was reduced to 50% of welds, at random. Welding inspection for the Garden structures was performed through radiographic tests on 57 individual weldments over a period of 24 days, through magnetic particle tests encompassing four days and through dye-penetrant tests covering 20 days.

It should be noted that this new Brooklyn Botanic Garden facility, in which the architecture and structure are so totally integrated to be one and the same, owes its successful completion to the cooperation of all the construction forces to conform to the design—the general contractor, steel fabricator, erector, architect and structural engineer and the welding inspector. Full cooperation in quality control was extended by the owner and the various contractors and fabricators throughout the work. For a project such as this new Conservatory, where an extraordinary result is asked of the professional and contracting teams, success can only be judged by how well the owner's criteria are met.

Brooklyn Botanic Garden President Donald E. Moore said, "We asked our design and construction team for a special new Conservatory to attract more visitors to the Garden and help us show them some of the wonders of the botanic world. Thanks to the personal dedication of each member of the team, we got those special buildings. We believe they will help people to better understand and delight in the plant kingdom." □

Architect
Davis, Brody & Associates
New York, New York

Structural Engineer
Goldreich, Page & Thropp
New York, New York

General Contractor
The DeMatteis Organization
Elmont, New York

Owner
Brooklyn Botanic Garden

Erection complete on one cross space-truss frame.

Joseph D. Goldreich is a partner in the consulting engineering firm of Goldreich, Page & Thropp, New York, New York.

Steel Notes

1988 AISC FELLOWSHIP WINNERS NAMED

Five engineering students have been named winners of 1988 graduate fellowship awards by AISC's Education Foundation. They receive individual $8,000 grants, to be shared with their schools, to support graduate studies. Winners were chosen on the basis of scholastic record, faculty recommendation and prospective benefits of the proposed graduate study to the engineering profession and the structural steel industry. The 1988 winners are:

- **Theodosios Heotis,** Illinois Institute of Technology. Development of a method for deriving a multi-story eccentrically braced frame configuration which optimizes both safety and economy in resisting both earthquake and wind loads.
- **John P. Van Leeuwen,** Oklahoma State University. Determination of the effect of heat straightening on material properties and assessing the influence of any changes in properties on member performance.
- **Paul B. Hofland,** University of Colorado at Boulder. Development of a valid, practical method for analysis of flexibly connected steel frames, suitably verified, for use in the design office.
- **Alan R. Huntley, Jr.,** University of Massachusetts. Development of computer procedures to determine the effect of using actual member lengths on the lateral deflection of rigid steel framing.
- **Barry K. Arnold,** Utah State University. Determination of the most effective type of moment connection when connecting a wide-flange girder to tube steel columns.

BRIDGE COMPETITION JUDGING TO BE HELD IN AUGUST

Judging for the 1988 Prize Bridge Competition will take place Aug. 9 at AISC headquarters in Chicago. Jurors will choose the most outstanding steel bridge designs which use structural steel aesthetically, economically, imaginatively and effectively. This year's four jurors are: **Dr. Lynn S. Beedle,** professor of civil engineering, Lehigh University; **Albert A. Grant,** president, American Society of Civil Engineers; **Dr. Arthur W. Hedgren, Jr.,** vice president, HDR-Richardson Gordon, Inc., Pittsburgh; and **John L. Smith, Jr.,** state bridge design engineer, North Carolina DOT, Raleigh.

Winners of the competition will be honored at the Eighth Annual Awards Banquet to be held Dec. 1 at the Westin Hotel, Chicago.

SPECIAL SALE OF AISC PUBLICATIONS

AISC is making a special offer on technical manuals. One copy of *Engineering for Steel Construction* (M014) is available for $10 with each purchase of a *Load and Resistance Factor Design (LRFD) Manual of Steel Construction* (M015), 1st Ed. or *Manual of Steel Construction* (M011), 8th Ed. *Engineering for Steel Construction regularly* sells for $39 to Institute members, $52 to non-members, the *Manual of Steel Construction* is $36 to members, $48 to non-members and the *LRFD Manual of Steel Construction* is $42 to members, $56 to non-members. The sale continues through Nov. 1, 1988.

To take advantage of this price break, send check, money order or Visa/MasterCard information (state type of card, number and expiration date) to AISC Publications Dept., P.O. Box 806276, Chicago, Ill. 60680-4124. All sales are final.

BOLT SPECIFICATION REMINDER

AISC reminds engineers and practitioners that the *Specification for Structural Joints Using ASTM A325 or A490 Bolts* is available for sale. The Specification, approved by the Research Council on Structural Connections of the Engineering Foundation, includes new provisions for "snug-tight," high-strength bolted connections. It defines material and shipping requirements, effects of overspray, proper use of installation methods and effective installation. The 48-pg. Specification (S329), endorsed by AISC and the Industrial Fasteners Institute, is $3.00 to Institute members, $4.00 to non-members.

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SOUTHEAST FINANCIAL CENTER

A Celebration in Structural Steel

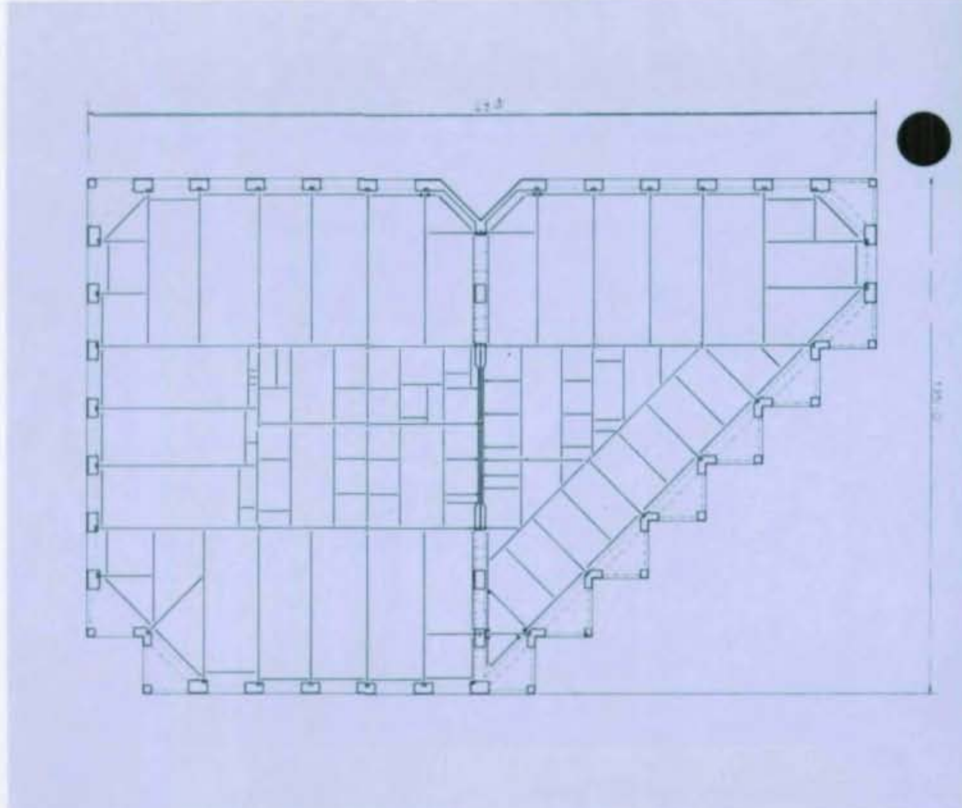
by Robert A. Halvorson and Robert Armsby

The Southeast Financial Center, the new headquarters for the Southeast Bank in Miami, Fla., consists of three elements: a 55-story tower with 1.2 million sq. ft fronting on Biscayne Boulevard; a separate 15-story annex containing the banking hall; 12 levels of parking for 1,200 cars, a retail arcade and an athletic club, with a large landscaped, open-air court beneath a space frame between the buildings.

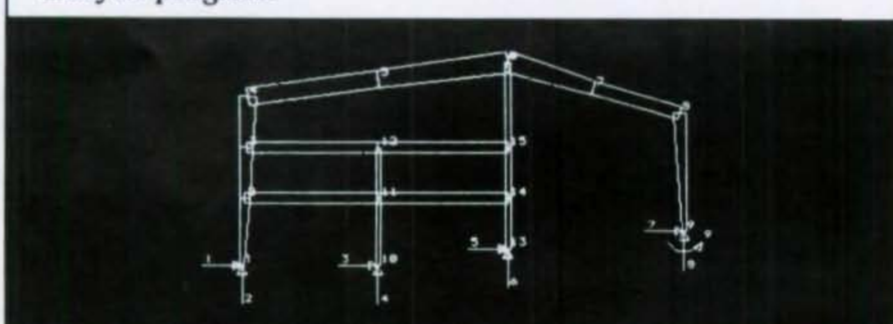
The tower form is basically rectangular. However, the southeast corner of the building steps back in a series of bays turning away from a near neighbor and orienting to the park and bay.

Both buildings are clad in a white, thermal finished granite. The tower elevations have a grid overlay of black polished granite strips into which a large window is introduced. In turn, it is divided into four equal lights, all framed in white and glazed with silver reflecting glass to create an elaborate pattern of squares within squares. The annex employs the same materials (except for glass) in a different manner, incorporating pierced openings to facilitate garage ventilation.

Both the tower lobby and the banking hall are connected by a glazed bridge one floor above the plaza. Retail space is accessible from the southeast Third Avenue side of the plaza from where pedestrian traffic is generated. The plaza is roofed by a white steel space frame 12 stories above its surface and is paved in a pattern of stones and tiles shaded by royal palms. The plaza is furnished with benches, tables, chairs and other accoutrements of public space.



Typical floor plan of Southeast Financial Center



Two Factors in Structural Concept

The selection of a structural concept for the tower was primarily influenced by two site-related factors:

Wind climate: Miami is located in an area subject to extreme tropical storms and its terrain subjects buildings to the full brunt of such storms.

Foundation conditions: Subsurface conditions in Miami are very difficult, so it was essential to develop a concept minimizing the overall weight of the tower.

After study, it was determined that a composite scheme, using both structural steel and reinforced concrete would best meet these site constraints as well as other owner requirements. A typical floor plan reveals the general concept: a bundled-tube system with reinforced concrete frames around the perimeter and through the center of the tower along an east-west centerline; all interior floor framing and core columns of structural steel. This system had a number of advantages:

1. All wind loads are resisted by reinforced concrete sections which provided necessary mass, damping and strength to resist the extreme wind loads (wind base shears were comparable to those of the Sears Tower in Chiçago, which is twice as tall!). In addition, this concept permitted all structural steel to be designed with simple

gravity shear connections, thus minimizing its cost and erection time.

2. Floor framing and interior columns were of lightweight structural steel to reduce as much as possible the overall weight of the tower and therefore its foundation requirements.
3. Additionally, structural steel provided all the benefits commonly found for structural steel framing, i.e.:
   - Flexibility to adapt floor framing for tenant needs—including higher loads and stair openings
   - Minimal column sizes within the core, and
   - Ability to adapt readily to setbacks in tower massing.
4. Lastly, the desire to retain one advantage of a pure structural steel frame even with the composite system—erection speed. Consequently, light steel erection columns were placed at all primary tube columns locations. With these, the steel frame was erected first (using temporary bracing) and the reinforced concrete perimeter spandrel beams and columns followed steel erection by some 8 to 12 stories.

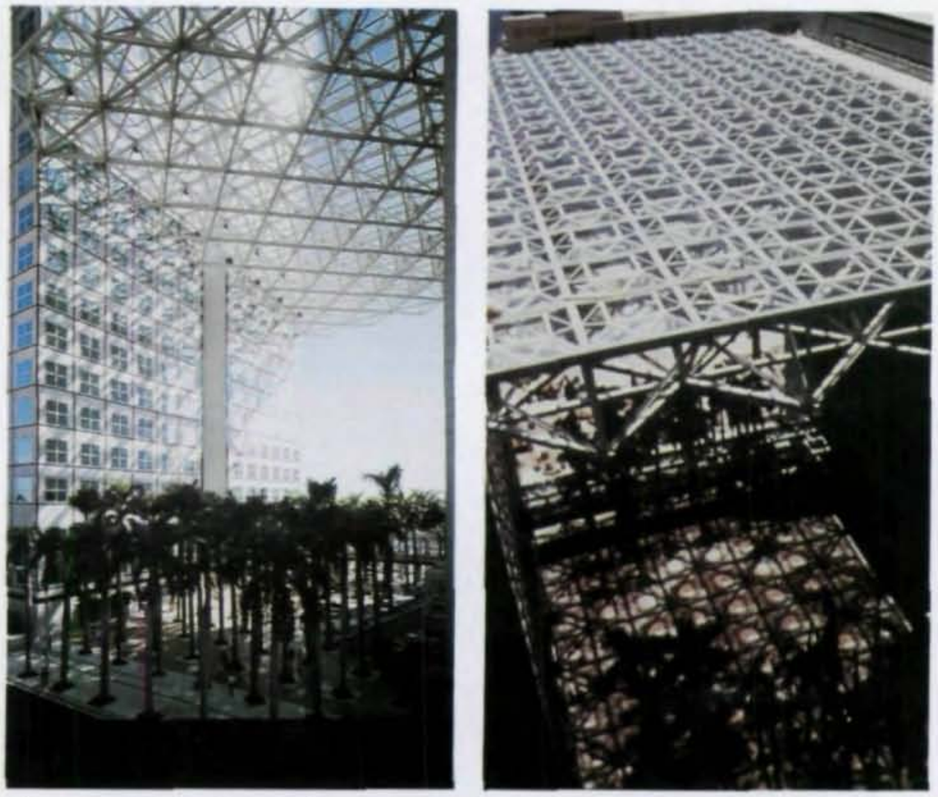
However, along with these advantages came some complexities in design:

First, that of compensating for the differing characteristics of the materials used in vertical elements, and

Second, in evaluation of the tower's dynamic properties for wind tunnel testing.

Structural steel is an elastic material, where concrete is subject to long-term deformations due to creep and shrinkage. While it is relatively simple to calculate final shortenings of columns and thereby provide the contractor with theoretical elevation corrections to compensate for differential shortening, this information is of little or no use in the field where achieved tolerances are significantly larger than such corrections. Therefore, SOM, in concert with Mark Fintel, developed a methodology and corresponding computer software to calculate the relative elevations at which steel should be placed *during erection*. A testing program was also devised to measure elastic, creep and shrinkage properties of the actual concrete mix designs. This work resulted in achieved floor elevations generally within ± 1 in. from average.

In calculating lateral dynamic properties of the tower, several finite-element studies were conducted on typical frame joint configurations to determine effective shear stiffnesses of the monolithic concrete frame. Such effects were significant since columns were 5 ft wide at 15 ft o.c. and beams were approximately 4 ft deep in



Magnificent 12-story high steel space frame, painted white, covers pedestrian plaza.

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floor heights of 13 ft-3 in. It was found the effective stiffnesses of frame elements was increased approximately proportional to the centerline span divided by the clear span, e.g., increased perhaps 50%. Calculated and measured lateral periods given below indicate very good agreement and reveal the very stiff nature of this composite system.

	Mode 1 N.E. to S.W.	Mode 2 N.W. to S.E.	Torsion
Calculated-sec	3.7	3.2	1.8
Measured-sec	3.5	2.9	1.8

Centerpiece in Steel

The centerpiece of the project is a celebration in structural steel—the space frame over the central courtyard. It is about 230 ft wide and spans 120 ft between the tower and the annex building, 170 ft above the plaza. The space frame design was conceived in rolled shapes using connection nodes fabricated from steel plate. The top chords of the space frame follow the rectangular grid of the site and frame 15-ft × 15-ft openings for skylights. The bottom chords were rotated 45° to the primary grid, a pattern reflected in the skylight and plaza geometries. For stability, members were inserted in the plane of the bottom chords following a rectangular grid around the perimeter of the space frame. The truss was erected in three sections (north, middle and south) on the ground, then each section hoisted into place.

The completed project employs concrete and steel in accordance with their peculiar natures: concrete, where the requirements include mass, damping and continuity; steel where the needs are lightness, compact dimensions and flexibility. □

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Robert Armsby is an architect and a partner of SOM. He served as administrative partner on this project.

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