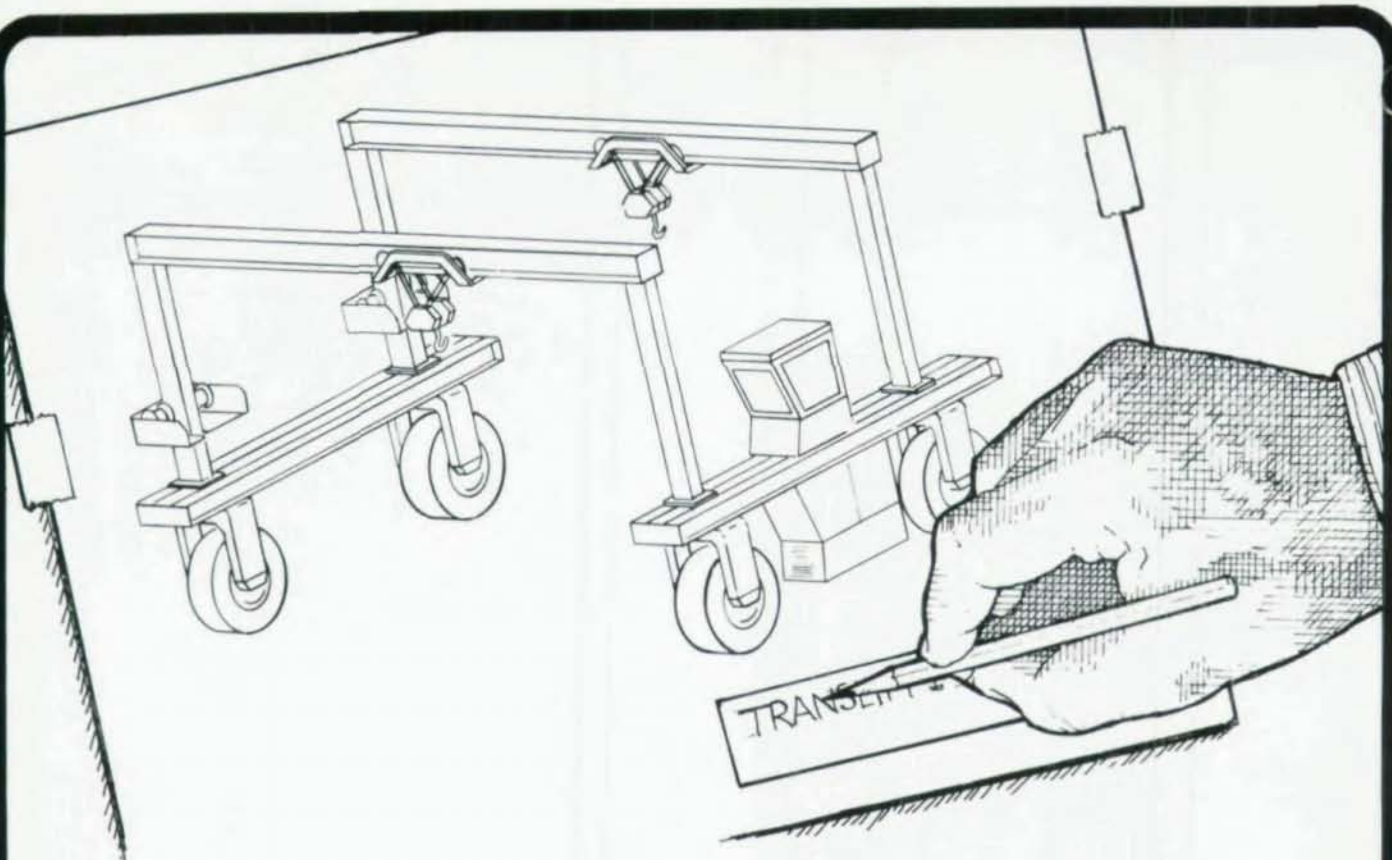
VOLUME XXIII NUMBER 2/SECOND QUARTER 1983

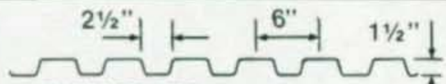
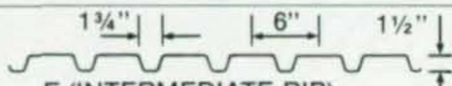
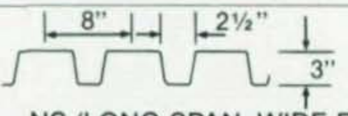
MODERN STEEL CONSTRUCTION



DECK DESIGN DATA SHEET

NO. 2

MAXIMUM ROOF DECK SPANS

UNITED STEEL DECK, INC. ROOF DECK PROFILE	DESIGN METAL THICKNESS	GA.	MAXIMUM SPAN		
			SINGLE SPAN	MULTIPLE SPAN	CANTILEVER SPAN
 B (WIDE RIB)	0.0295"	22	5'-10"	6'-11"	2'-0"
	0.0358"	20	6'-7"	7'-9"	2'-4"
	0.0474"	18	7'-11"	9'-5"	2'-10"'
	0.0598"	16	9'-0"	10'-8"*	3'-0"
 F (INTERMEDIATE RIB)	0.0295"	22	5'-1"	6'-1"	1'-6"
	0.0358"	20	5'-9"	6'-9"	1'-9"
	0.0474"	18	6'-11"	8'-2"	2'-3"
 NS (LONG SPAN, WIDE RIB)	0.0295"	22	11'-6"*	13'-6"*	4'-0"
	0.0358"	20	13'-0"*	15'-4"*	4'-7"
	0.0474"	18	15'-9"*	18'-7"*	5'-5"
	0.0598"	16	18'-3"*	21'-6"*	6'-0"

*Exceeds normal applications

NOTES:

All maximum spans are center to center and are based on SDI loading criteria and United Steel Deck, Inc. roof deck sections.

1.) Regular spans (not cantilever) are governed by a maximum stress of 26600 psi and a maximum deflection of 1/240 with a 200 pound concentrated load at midspan on a 1'-0" wide section of deck.
2.) Cantilever spans are based on:
   a.) construction load of 10 psf on adjacent span and cantilever, plus 200 pound load at end of cantilever - stress limit is 26600 psi; or
   b.) service load of 45 psf on adjacent span and cantilever, plus 100 pound load at end of cantilever -stress limit is 20000 psi and cantilever deflection limit is 1/120.
   c.) maximum, and less than maximum, adjacent spans were used to find the cantilever spans; the governing shorter spans are shown in the table.
3.) Check any applicable insurance requirements (Underwriters Laboratories and Factory Mutual) as they may require smaller spans.
4.) Uniform loads are shown in the U.S.D. catalog for spans greater than the maximums shown in this table. Frequently deck is used in applications other than roofs - siding, temporary structures, shelving, etc., and load data is desired.
5.) Reprints available on request.

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CONTENTS

1983 FELLOWSHIP AWARD WINNERS NAMED

This year's eight winners of the AISC 1983 Fellowship Awards competition have been named. Each receives a $4,000 study fellowship, with $750 additional going to their academic department heads to administer the program. They were judged by an award jury on the basis of grade-point average, faculty recommendations and the prospective contribution in their study programs to the engineering profession and the structural steel industry.

The 1984 winners are:

Donald D. Cannon, Jr., *University of Tennessee-Knoxville*
Daniel P. Clark, *Portland State University*
Larry E. Curtis, *University of Oklahoma*
Kenneth R. Hamm, Jr., *University of Arizona*
Andrew W. Taylor, *University of Washington*
Pedro M. Vargas, *University of Kansas*
Katherine A. Wellspeak, *Rensselaer Polytechnic University*
Donald W. White, *Cornell University*

1983 T.R. HIGGINS LECTURESHIP AWARD TO OMER BLODGETT

Omer W. Blodgett, a design consultant for Lincoln Electric Company, has been selected as the 1983 winner of the T. R. Higgins Lectureship. His winning paper—"Detailing to Achieve Practical Welded Fabrication"—appeared in Engineering Journal, *4th Quarter 1980. An engraved citation and $2,000 was awarded to Blodgett at the 1983 National Engineering Conference in Memphis.*

"VOLUME II" SUPPLEMENT IN BD&C FEATURES STEEL

For the third successive year, the "Volume II" supplement of the June Building Design & Construction *features the use of steel and its structural advantages in buildings. Six major U.S. structures are featured in the 48-pg. supplement. Individual copies may be secured from: AISC Member Services, 400 N. Michigan Ave., Chicago, IL 60611. Price: $1.50 per copy.*

NEW DETAILING FOR STEEL CONSTRUCTION AVAILABLE

*AISC's new textbook—*Detailing for Steel Construction*—will be available on July 31. The new book—prepared as a guide for school or on-the-job training of structural detailers—contains up-to-date information, examples, sample drawings and reference data keyed to the needs of the beginning detailer or student who has a basic knowledge of math or trigonometry and a high school-equivalent education. The 288-pg. book, cross-referenced to the 8th Edition* Manual of Steel Construction *is available from AISC Book Dept., Box 4588, Chicago, IL 60680. Price: $32.*

Four Allen Center:
New Concepts in the Wind

by James S. Notch

The structural analysis and design process as it evolved in planning the new Four Allen Center Tower in Houston, Tex. is most interesting. The $160-million tower is the latest addition to Allen Center, a $1-billion development in downtown Houston.

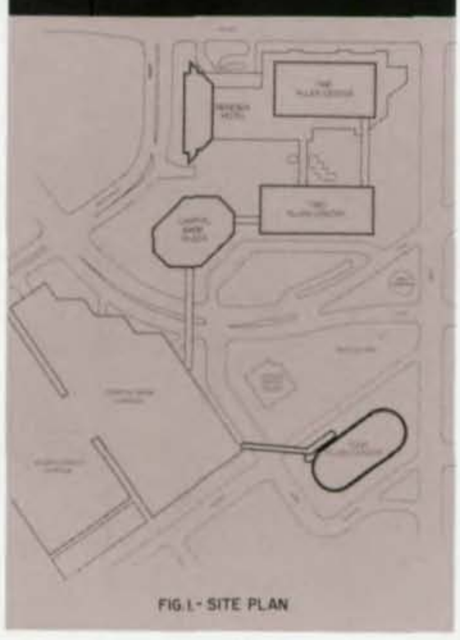
The 1.44-million sq ft office tower, with 50 above grade levels and two subgrade levels, rises 695 ft above street level. Surrounding the silver reflective glass tower, a subgrade structure contains mechanical areas, tunnel level shops, food service facilities and ingress-egress to the downtown Houston pedestrian tunnel system. Landscaped areas over the surrounding structure intermesh with an escalator lobby appendage which ties into the Allen Center skyway system (see Fig. 1).

The circu-ovalar building plan shape, or in simpler terms, an elongated rectangle with semi-circular ends, has overall plan dimensions of approximately 109.4 x 259.4 ft. The curved ends of the building, in conjunction with a 40-ft lease space depth from curtain wall to core, provides the owner with a flexible floor plan and offers renters panoramic views of Houston.

The narrow, elongated plan shape of this project created an exceptionally slender tower. With its horizontally banded reflective glass and white fluropon coated aluminum panels, the tower will create a distinctive "knife-edged" silhouette and an individual identity on the Houston skyline. The extremely slender tower, with a perimeter frame aspect ratio in excess of 6.85 and the constraint of a central shear truss core depth of only 25.75 ft, presented a formidable challenge to the structural engineers. Further compounding the engineering problems were results of an aeroelastic wind tunnel test on the building shape which predicted that extreme dynamic oscillations of the tower may occur under hurricane loading conditions.

To meet these challenges, an innovative hybrid structural system was conceived. A four-celled bundled framed-tube system was developed to meet the demanding strength and serviceability requirements. The perimeter frame was assembled from two-story high tree-column modules located at 15 ft o.c. around the building perimeter (see Fig. 2). The cross frames which subdivided the plan into its four-celled plan were formed by horizontal tree-beam modules interacting with diagonalized trusses in the shallow central core area (see Figs. 3, 4). The structural system, as it evolved, borrowed many recent structural engineering developments and molded them into a new, and unique, structural framing concept.

James S. Notch, P.E., is vice president of Ellisor & Tanner, Inc., consulting engineers, Houston, Texas



Four Allen Center Tower, Houston; with site plan (Figure 1)

The tree-beam continuum element concept, which introduced several columns in the lease area, was scrutinized at length by the space planner—and was further reviewed by the owner's marketing department, the financial partner's project evaluation staff and the project architect's space planners. These representatives agreed unanimously that, although the columns were psychologically troubling when viewing the wide open floor plan on paper, they created minimal loss of space-planning freedom. Many space-planning layouts were prepared to demonstrate that the columns could be integrated quite easily with most tenants' requirements.

Structural System Conceptual Development

Based on previous experience with multistory buildings, the architect knew the importance and economics of developing a structural system in which the entire building shape could be facilitated to resist Houston's hurricane level wind loadings. From the onset, the architectural designers incorporated closely spaced columns at the building perimeter with the hope it would provide the engineer with an efficient framed-tube structure. What the architects had not planned on, however, was that the framed tube in its pure form would be both structurally inadequate and inefficient for the particular building plan shape and positioning as configured. A conventional framed tube was inappropriate for several reasons:

1. Framed tube systems are ideally suited for compact plan-shaped buildings. To be truly efficient, the web portion of the perimeter framed-tube must include a sufficient number of column bays and possess sufficient flexural/shear stiffness to resist applied wind shear, and subsequently develop shear flow to activate axial force in the "flange" portion. The flange portion must also possess sufficient rigidity to minimize potential shear lag inherent in the system. The perimeter framed-tube, as originally configured and related to the building shape

Interior floor framing/tree column, with Two Allen Center in background

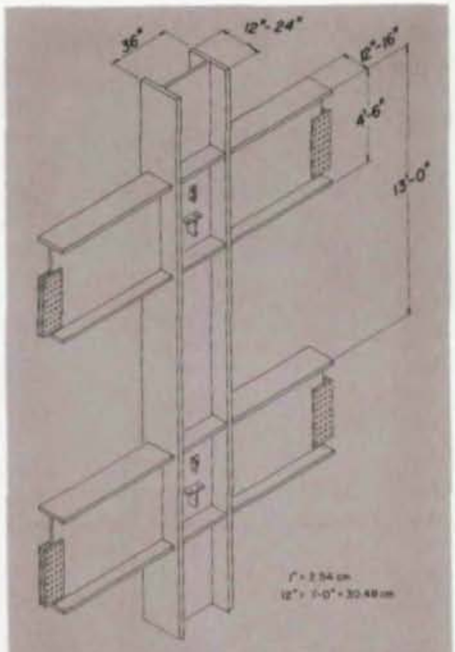


Figure 2a—Typical H-shaped tree column module

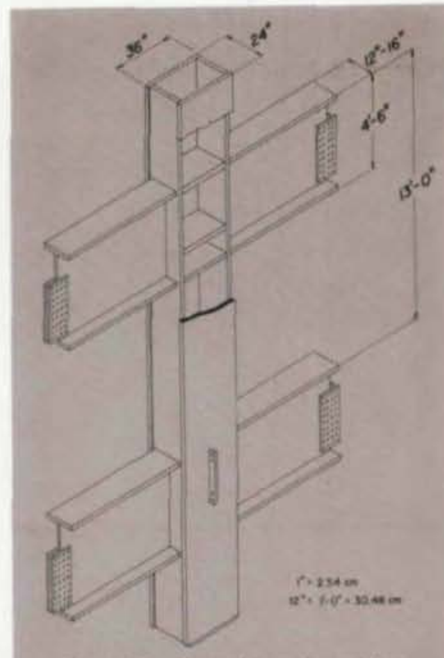


Figure 2b—Typical box-shaped tree column module

and site, did not meet these guidelines. The building shape as related to the rigidity of the perimeter frame was disproportionate—wind on the narrow exposure, with relatively small resultant wind forces, had the strong parallel frames working to resist the wind. Conversely, for wind from the orthogonal direction, i.e., wind on the broad exposure, the elongated building plan shape provided a broad surface area which acted like a large sail, thus accumulating large magnitudes of wind load. These large forces could not be resisted, since the along wind dimension of the structure was extremely narrow and offered little resistance to the wind effects.

2. Acting as a framed-tube structure and with respect to wind on the broad exposure, the web frames at the semi-circular ends of the building were very flexible. This was due not only to the short frame length but also to the fact that columns were located on a curve with an extremely tight radius of 52.71 ft. Even if the stiffness of the semi-circular end frames could be augmented, efficient tube action would have been difficult to achieve because of the great distance between end frames. The parallel perimeter "flange" frames, which interconnect the semi-circular web frames, were of such length that shear lag phenomenon would have been pronounced.

3. The building shape, oriented at a skew with the typical downtown grid system, and located directly downwind from a randomly placed series of 31- to 36-story structures, created an environmental condition in which wind speeds were greatly accelerated and modulated prior to striking the tower. This condition resulted in dynamic amplification effects which magnified by a factor of two or three the wind loads predicted, based on a static wind tunnel test. The magnitude of wind loading encountered required a unique structural approach.

In response to the aforementioned difficulties in selecting a pure perimeter framed-tube system as the structural system for the project, engineers and architects worked closely together in exploring all options. Their specific intent was to develop some type of hybrid system to provide a strong, serviceable structure, yet one which would maintain the well-defined goals of the architectural team.

Conceptual level computer study models were set up to aid in modifying the framed-tube system as originally configured into a workable solution. Prior to the hybridization process, the basic perimeter framed-tube was analyzed. Results confirmed the engineer's predictions. The frame was "mushy"—the curved end frames did not develop proper shear resistance nor activate the long inefficient flange frames. In addition to severe overstresses, the tower swayed more than the three times tolerable limits.

The first step in the system development was to add several lines of diagonal bracing (inverted K-truss type) within the core of the building. Even though flexurally weak due to the short truss depth of 25.75 ft, adding the shear trusses proved of benefit in providing a positive shear resistance medium for the building. Due to the semi-circular building ends, very little shear resistance would have been available without adding the diagonal shear bracing.

The problem of shear lag in the long rectilinear perimeter frames created a more complex challenge in the system development process. The first attempt to minimize shear lag concentrated on the use of outrigger trusses to link the trussed core with the perimeter frames. Conceptual studies clearly demonstrated, however, that using outrigger trusses in conjunction with the shallow core bracing was ineffective, regardless of the level or levels at which outriggers were placed. Core columns did not possess enough axial stiffness, acting at their respective distance from the building centerline, to provide the necessary rotational restraint for the outrigger element. Furthermore, since the tower was served by a remote central mechanical plant, the architect did not want or need the intermediate mid-height mechanical levels which could have accommodated conventional outrigger/belt trusses.

Structurally, some type of continuum element was needed to provide for continuous shear flow between core bracing and perimeter columns. The logical choice would be a moment-connected wind girder system between core and perimeter columns. Due to architectural constraints, it was feasible to introduce only three lines of frames across the tower. The structural studies performed demonstrated, however, that the resultant member size as required for the 40-ft long girder at the three wind frame grid lines was prohibitively heavy and not dimensionally compatible with the interstitial space available. To enhance the stiffness and strength of the wind girder concept, the tree-beam concept was adopted in which short vertical stub columns at mid-span were added to heavy horizontal wind girders (see Fig. 5). The stub columns forced an intermediate inflection point in the member, thus greatly increasing its stiffness and strength. Since the stub pieces performed only a flexural function, and since shear truss/frame action is dominant in the intermediate height range of the tower, they were easily deleted in the lower levels and top levels to create expansive, open architectural spaces. Through an active dialogue with the owner/developer, architect and space planner, the columns were carefully integrated into the space plan with little difficulty.

Fine tuning of the wind resistant system occured by adding two-story deep subgrade trusses at each of the three tree-beam/shear truss frames (Fig. 3). Computer stress analyses showed that extreme tensile uplift forces occurred at the base of the wind frame core columns located at the terminal end of each core area shear truss. The uplift force, which would have caused foundation design problems, was effectively translated to the building perimeter columns via the subgrade truss. The truss' presence forced the mat foundation to move as a unified element in resisting wind loads, rather than introducing concentrated moment couples at isolated locations on the mat.

Subgrade trusses were also beneficial in reducing the amount of differential settlement the mat would undergo. The trusses act like giant "strong backs" to resist mat curvature under short-term elastic and long-term consolidation settlement. Furthermore, they were beneficial in reducing frame translation at subgrade levels, thus minimizing joint problems with the rigid non-yielding plaza level during periods of tower translation due to wind. Also, due to reduced lateral translation at level B1 and 1 diaphragms, wind force in the lower level wind frame beams and columns was greatly reduced. This optimized the perimeter frame member sizing and resulted in a much more uniform design capacity in the three-story high first tier columns. The reduced spandrel beam forces at subgrade levels allowed the use of shallower perimeter beams, thus allowing passage for a multitude of subgrade piping and ductwork.

Due to the high aspect ratio of the tower (height/least width), its unusual shape and the presence of surrounding tall structures, the structural engineer recommended that both a static and an aeroelastic wind tunnel test be conducted.

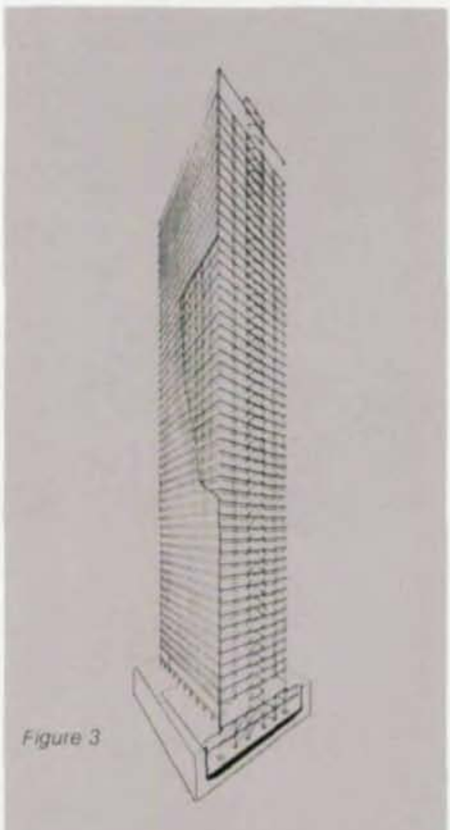


Figure 3

Dramatic exterior of steel erection (above). Note long-span pedestrian bridge truss in right foreground. Figure 3 shows overall building section.

Wind Drift Control

Wind drift control is an integral part of the structural design of any multi-story building. The need to limit building motion under lateral loadings can be categorized into three areas:

1. Structural stability.
2. Architectural integrity.
3. Occupant comfort.

1. *Structural Stability*

As the office tower deflects laterally, the mass centroid of each floor is displaced horizontally from its original location. This shift in mass generates destabilization forces which increase the tower's tendency to overturn in the wind. Based on the 100-year MRI wind tunnel forces (including dynamic effects), initial deflections of the office tower were calculated. Once these movements were known, effect of the mass shift of the tower was determined and applied wind tunnel forces augmented accordingly. This is based on recommendations of the Structural Stability Research Council as contained in the *Guide to Stability Design Criteria for Metal Structures.*

2. *Architectural Integrity*

Horizontal movement of a building due to wind loads may result in distress of the internal partitions and the external cladding of a structure. Mechanical, electrical and elevator sub-systems are similarly affected. It is important that proper drift criterion is developed by the structural engineer, acting in close harmony with other members of the design team. It is essential that all building components function properly with a relatively low probability of distress under extreme deflection conditions. To provide assurance that all building components were compatible with the anticipated motion of the structure, the estimated extreme movements of the structure (based on 100-year MRI wind) were documented on the design documents. By proper anticipation of the building's movements, all sub-systems have been designed accordingly to move with the structure and undergo minimal distress.

On buildings where custom provisions are not made in the detailing of architectural systems to accommodate building movement, the standard engineering practice is to limit drift to various drift indices which should, by themselves, minimize architectural system distress with minimal special detailing. In reviewing a structure's conformance with commonly accepted drift indices, accepted practice among the engineering profession is to review serviceability criteria based on a reduced recurrence interval wind from that used for strength

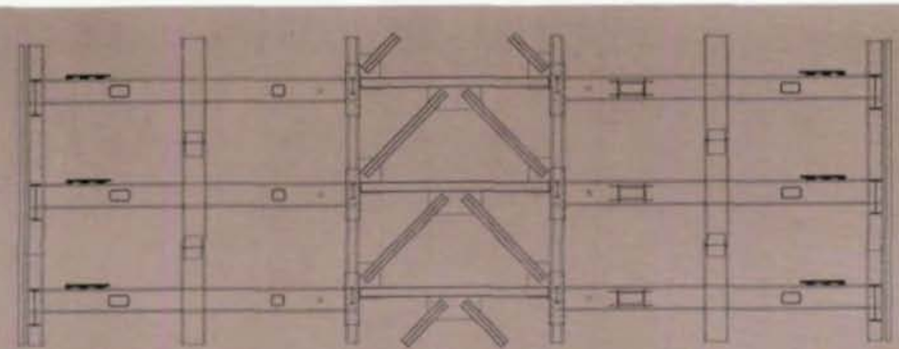


Figure 4—Enlarged building section on typical floor

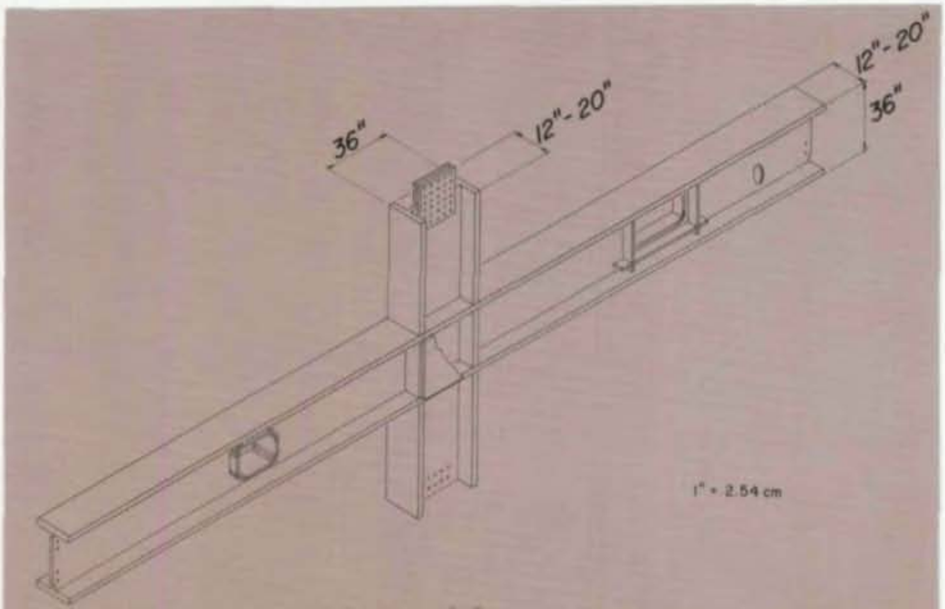


Figure 5—Typical tree column module

design. It was the engineer's judgment to use 50-year MRI winds for serviceability design.

It should be pointed out, the high wind loads as reported by the wind tunnel study were primarily dynamic in nature and resulted from wind gusts of very short duration (about five seconds). To assure that the safety and integrity of the structure be maintained under extreme environmental conditions, it was necessary that wind tunnel consultants be conservative in preparing their data. In reviewing drift criteria, we may rationalize that a longer duration gust could be used, thus reducing wind forces and resultant deflections. Due to tight project schedules, this beneficial correction was *not incorporated. However, its potential* effect was considered in evaluating the wind drift criteria.

The components of wind drift for the controlling azimuthal direction are:

Maximum 50-yr MRI Wind Drift (with non-load factored secondary moments) (Azimuth 110° node located at extreme curved end of building, roof level)

Direct shear components:
X disp = 17.00 in. (431.8 mm) Y disp = 1.41 in. (35.8 cm)

Torsion components:
X disp = 4.08 in. (103.6 mm) Y disp = 0.24 in. (6.1 cm)

Total components (quadrant 4 critical):
X disp = 21.08 in. (535.4 mm) Y disp = 1.65 in. (41.9 cm)
Resultant Deflection = 21.14 in. (537.0 mm)
21.14/(708 x 12) = 1/402 (or) 0.0025 overall drift index

3. *Occupant Comfort*
Four Allen Center is a very slender office tower with a frame aspect ratio (ratio of height to least width) in excess of 6.85. This slenderness, in conjunction with the tower's shape and orientation, results in a project subject to significant dynamic excitation by wind forces. Based on the engineer's recommendation, an aeroelastic wind tunnel model was instrumented to predict the dynamic forces related to oscillation of the tower by the gust action of hurricane force winds. Based on the results of the aeroelastic tunnel test, it was the wind tunnel consultant's task to evaluate the magnitudes of drift and dynamic accelerations and comment upon their acceptability for occupant comfort.

Peak horizontal acceleration was observed to be about 0.07 meter/sec^2 for a 10-year return period wind. This value occurred within the commonly accepted range of 5 to 15 milligs. The magnitude of *wind drift was also within commonly* accepted levels for occupant comfort.

A fine line exists in the engineering profession regarding what constitutes acceptable drift criterion. The question goes beyond the simplistic selection of a proper drift index. It is dependent on many variables, such as how the wind loads were obtained, assumptions used in the analysis, etc. Further, it is dependent on the specifics of the individual project such as the types of partitions, curtain wall connection details, etc. Based on the engineer's total involvement over many months in the project's design/development, a structural system was evolved which not only fulfilled strength and stability criteria, but also met high standards for serviceability criteria.

Primary Structural System

The perimeter framed-tube system typically consisted of two-story high tree-column modules approximately 15 ft o.c. (see Fig. 2). Of the 42 perimeter column shaft locations, 36 were of an H-shape configuration, while the six columns located at junctures with the interior wind frame bents were built-up as box-shaped sections.

The H-shaped columns consisted of three plate weldments on the lower half of the tower and transitioned to W36 rolled shapes at the upper half of the tower. The H-shaped weldment had flange plates varying in size from 4½ in. x 24 in. to 1-3/4 in. x 20 in. and web plate thicknesses varying from 2½ in. to 1 in. Built-up H sections varied in weight from 964 plf to 349 plf. The rolled column sections varied in size from W36x300 to W36x135. The built-up column web-to-flange welds were sized as required for various stress level conditions. In the locations *between spandrel beams, shear flow stres*ses were relatively low and an AWS minimum continuous fillet weld was generally sufficient. In the area within and adjacent to the beam-column joint, very high shear forces existed and made it necessary to provide a partial penetration weld with continuous fillet weld overlay. This additional weld was also needed to transfer spandrel beam flange forces through the column flange into the web.

The column shafts located at the juncture with the three interior wind frames were configured as box sections so that significant strength and rigidity would be available about both axes. The outside dimensions of the box section were maintained at the constant value of 36 in. in a direction parallel with the building perimeter and 24 in. in a direction perpendicular to the perimeter building line. The box shape column was configured as a four-piece weldment, with plate sizes varying from 6½ in. thick at the base to 3/4 in. thick at the top tiers. Column shaft weight varied from 2,078 plf at the base to 298 plf at the top. Similar to the prefabricated H-shape weldments, welds near the panel zones were increased in size to resist the biaxial effect of shear flow transfer in two directions.

Window wall attachments were typically located adjacent to floor slab diaphragms to minimize minor axis bending of the perimeter column sections. At level 50, with a 27-ft floor-to-floor height, and at level 1 with a 25-ft floor-to-floor height, window wall design economics dictated placement of an intermediate level window wall lateral support. The affected columns consequently were designed for the generated minor axis bending.

Column-to-column splice connections were made midway between spandrel beams at points of theoretical minimum moment. Typical flange welds were partial penetration to a depth of $\sqrt{T/6} + 1/8$ in. where T = column flange plate thickness. Weld depth was increased as required for columns with calculated uplift forces. Welds were increased at isolated locations to accommodate shifts in the inflection point of the moment diagram. All bearing surfaces were milled. Column web-to-web connections consisted of bolted double splice plate connections designed to resist the shear force in the column web.

Column web stiffeners aligning with the spandrel beam flanges were provided to meet requirements of strength, or in most conditions, to stiffen the beam column joint and restrain panel zone flexibility.

At most wind frame column bases, uplift forces due to lateral loading were significant. In some specific cases, such as at some of the perimeter box columns, the magnitude of uplift force at a column exceeded 2,975,000 lbs. Uplift forces were resisted by anchor bolt assemblies which were embedded deep into the base of the mat foundation. The 2-in. and 2½-in. diameter A354 grade BD anchor bolts were restrained from pull out by large stiffened washer plate assemblies.


Diagonal bracing connection with core column

Spandrel beam stub pieces, shop-welded to the perimeter columns to form the tree-column assembly, were typically 4 ft 6 in. deep. Spandrel beams consisted of three plate H-shape weldments with flange plates varying from 2½ in. x 16 in. to 3/8 in. x 12 in. Web plates varied in thickness from 1 in. to 5/16 in. Level 2 was non-typical, with spandrel beams 12 ft 3 in. in depth. Subgrade spandrel sections at plaza level and B1 level were of varied depth to respond to architectural and mechanical conditions. Built-up beam web-to-flange welds were automatically welded using double fillet welds to resist applied stresses.

Based on the beam flange thickness, beam flange-to-column flange welds consisted of either partial penetration welds with fillet weld overlays or fillet welds only. Beam web-to-column welds were double fillet welds. Beam-to-beam field splices were made midway between columns at points of theoretical minimum moment using 1-in. A490 bolts in friction type connections. Double shear values could be used by providing a splice plate on each beam web face. Oversize holes (bolt diameter + 3/16 in.) were used in these connections to facilitate field alignment during erection.

The internal cell partitions were formed using shop fabricated tree-beam modules along grid lines 2, 6 and 10 spanning from the perimeter box columns as previously described to internal wind frame columns located at the terminal ends of the diagonal core bracing (Fig. 5). The horizontal element of the tree beam was a 3-ft deep three plate H-shape weldment in the lower one-half to two-thirds of the tower height. Flange plates varied in size from 3-3/4 in. x 20 in. to 1-3/4 in. x 18 in. Web plates varied from 2-1/8 in. to 1 in. in thickness.

The tree-beam elements were fully moment connected using field welding to the columns at each end. The stub column piece to stub column piece connection joints were gapped and non-bearing with field bolt up using 1¼-in. A490 bolts with double splice plates. Connections at all odd floor levels were vertically slipped to relieve force build up associated with axial shortening of core and perimeter columns. Due to the magnitude of design wind force, some of the bolt groups at the lower fixed end of the slip connection were supplemented with field welds located at perimeter of splice plates.

Weight of the resulting fabricated sections ranged from 716 plf to 325 plf. In the upper regions of the building, rolled sections were used which varied in size from W36xW300 to W36x135.

Vertical stub column piece elements located at the center of the horizontal beam were made from rolled W36 shapes or three plate fabricated weldments. The stub column piece sizing was slightly less than its respective horizontal beam piece.

The vertical stub column elements were shop moment-connected to the horizontal member. Because of extreme panel zone stresses, thick doubler plates were added between flanges over each panel zone area. Since beam depth was optimized and extended tight to the ceiling levels, penetrations for mechanical ductwork and sprinkler piping were incorporated.

For fabrication simplicity, the inverted K-truss type diagonal bracing in the core area consisted of 2L or 4L struts which were lapped at each end and bolted to gusset plates. The horizontal strut consisted of a double channel with the gusset plates sandwiched and bolted in between at each end and the center. Gusset plates were simply fillet welded to the core columns as required. Fabrication/erection of the subgrade truss as previously described was more complex. Due to the magnitude of the forces, large rolled W14 shapes were used for the diagonal and horizontal truss components. Member sizes ranged from W14x90 to W14x426. Bolted double-lap plates were used at the web and flange areas to transfer design axial forces.

Interior Gravity Framing System

The typical floor construction consisted of 6¼-in. thick composite metal deck slab construction (3¼-in. lightweight concrete slab on 3-in. deep metal deck). The deck, typically spanning 15 ft, was supported by W21 rolled sections. At the curved ends of the plan, where deck span varied greatly, several deck gauges were used as required to economize the system.

Spanning the approximately 40-ft distance from the perimeter tree-column frame to the central core area, typical beams were designed to act compositely with the slab through shear studs, field-installed through the metal deck. A shop camber was specified on all long beams to compensate for the deflection of the beam under the weight of the wet concrete, thus providing a constant thickness and level floor system after pouring of the slab. Because of metal deck deflection between beams, the composite slab was thicker in the area between beams. The additional concrete ponding weight was considered in the gravity framing and deck design. Nominal 6x6-W1.4xW1.4WWM was provided in the floor slab. This slab reinforcement was augmented in several areas of high diaphragm stress as required. U-shaped rebar ties were provided at the perimeter of the building slab to provide a mechanical tie between the shear connectors located on the spandrel beams and the floor diaphragm. This mechanical tie provided for bracing of the columns into the floor diaphragm as well as transfer of wind shears into the diaphragm.

Due to the nature of a framed-tube system, it was important to achieve a framing configuration which would load the perimeter columns at a relatively uniform gravity stress level. Any large difference in the distribution of perimeter frame gravity loading would induce differential axial shortening between the closely spaced columns, thus generating large resisting moments in spandrel sections.

To achieve a relatively uniform column loading, typical floor beams were framed perpendicularly into each perimeter column. This created a series of parallel floor beams at 15 ft. o.c. along the parallel building faces and a series of beams in radial formation along the curved building faces.

Core columns were typically W14 rolled sections. In the lower tower levels where design loadings exceeded the capacity of a W14x730 section, cover plates were added which connected from flange tip to flange tip. The fabrication of welded box sections, 18 in. x 24 in., was required in some core locations to integrate with elevator and architectural system requirements.

At gravity column splices, flanges at milled bearing surfaces were welded with minimum partial penetration welds. Webs were selectively welded as required for shear. Weld size was increased as required in any locations where concentrated moments were applied to the column. To facilitate erection, all interior gravity columns were spliced at 2 ft 6 in. above finished floor, a height convenient to the steelworker.

Steel erectors at upper core level

All interior columns bear on milled steel base plates. The largest plate weighed 2,525 lbs. and was 52 in. x 52 in. x 9 in. A 3-in. thick layer of non-shrink grout (7,500 psi at 28 days) was provided between the bottom of the steel base plate and top of the concrete mat foundation. The base plates were flow-grouted from the side of the base plate assemblies, with no grout holes required.

To provide some contingency in the capacity of the interior columns to support future anticipated tenant file areas, computer rooms and other load conditions in excess of normal office buildings, a surcharge loading of 35 psf was added at level 50 and a surcharge loading of 5 psf at all typical lease levels. These surcharge loadings were used only for column design. During tenant work, composite beams can normally be stiffened for the increased loading by simply adding bottom flange cover plates. Stiffening of interior columns, however, can be extremely expensive because of the large number of levels affected and the difficulty in gaining access to the columns to add stiffening plates.

Careful consideration was given to the effects of axial shortening and foundation deformation on the detailing length of interior columns. Since most interior columns were designed of high-strength steel for gravity loading only, they were subject to much more axial shortening under gravity forces than the A36 perimeter columns designed as beam columns, subject to gravity and wind loading. Also, the mat foundation subject to the tower loading "dishes," thus lowering the interior columns relative to the perimeter columns. To compensate for these effects, the interior columns were detailed over-length. Column lengths were proportioned to achieve a level floor datum at one year after completion, since the soil engineer estimated that 80% of long-term settlement would take place by that date.

Foundation System

Unlike most other cities where tall buildings are supported on incompressible bedrock underlying the surface, Houston structures are founded on very thick deposits of compressible clay. The stiff clay deposits are primarily over-consolidated to a depth of 100 ft and change in nature to normally consolidated at depths in excess of 100 ft. In response to these soil conditions, most major structures in the Houston area bear on large concrete mat foundation systems.

The mat foundation under Four Allen Center approximates the plan shape of the tower, with an overall length of about 300 ft and a width from 150 ft to 154.21 ft. Mat thickness varies from a basic dimension of 8.5 ft to 19.25 ft at the service elevator pit areas. The mat foundation, reputed to be the largest single mat pour ever, contains 13,308 cu. yds. of concrete. Careful attention was given to the monitoring/limiting of concrete temperature during the pour and to sufficient provisions with steel reinforcement for resisting shrinkage/temperature cracking.

An intricate array of rebar trusses were installed below all column locations which had significant wind uplift forces. Through careful detailing, the engineer made sure that local flexing of the mat would not occur. The goal of forcing the mat to move and tip as a monolithic element when subjected to wind loads was accomplished. □

Architect
Lloyd Jones Brewer Associates
Houston, Texas

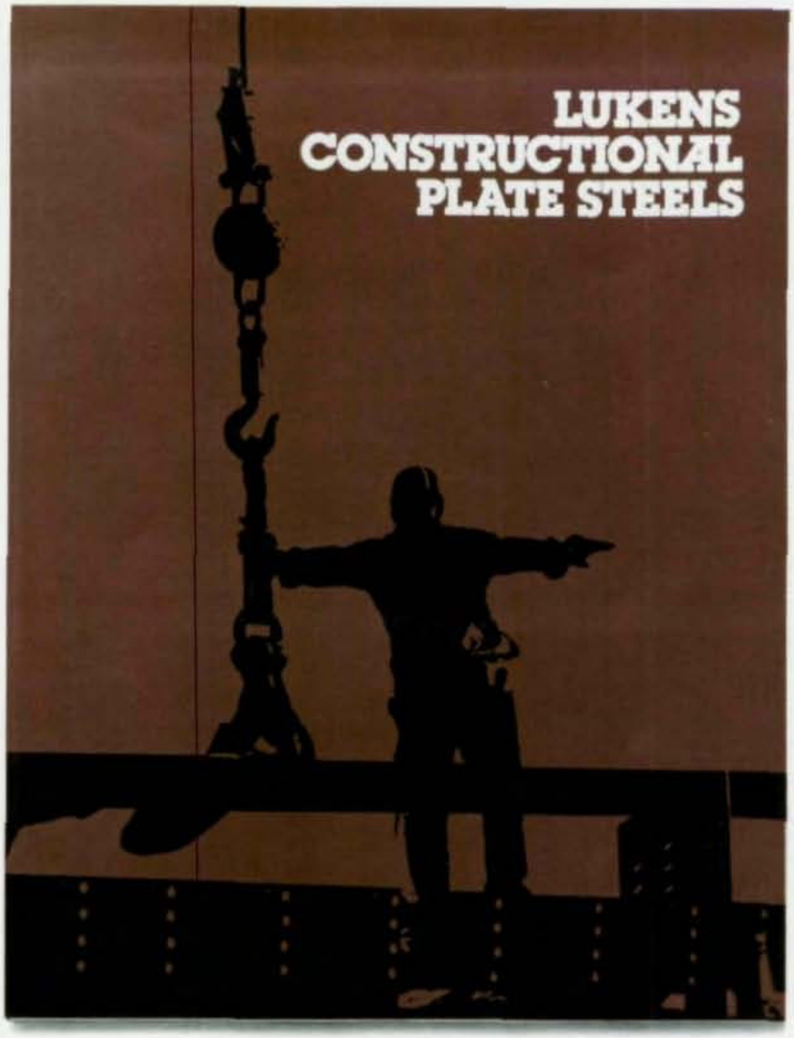
Structural Engineer
Ellisor & Tanner, Inc
Houston, Texas

General Contractor
McGregor Construction Co. (shell contractor)
Texas Construction, Inc. (interior contractor)
Houston, Texas

Steel Fabricator
Mosher Steel Company
Houston, Texas

Steel Erector
American Bridge Division, USS Corp.
Houston, Texas

Owner/Developer
Allen Center Company
A joint venture of Century
Development Corporation, Centennial
Equities Corp. (subsidiary of Metropolitan Life)
and American General Realty Co.
Houston, Texas



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Steel Trusses Going Up . . . Hospital Still Operates!

by Pradeep Patel

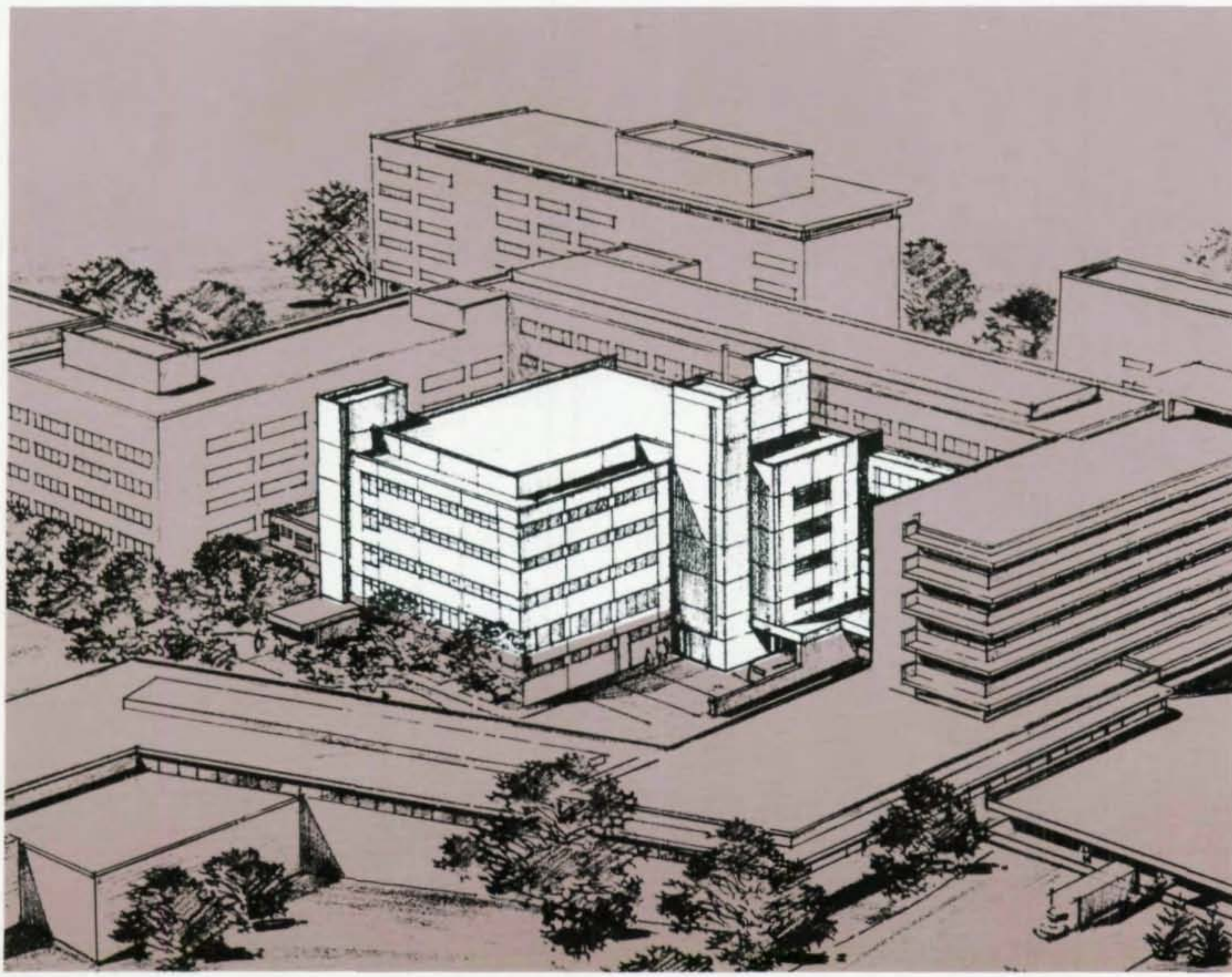
Pradeep Patel, P.E., a principal at Ewing Cole Cherry Parsky, directs the structural engineering department of the Philadelphia-based architectural/engineering firm.

How do you keep a suburban Philadelphia hospital in full operation while you build a four-story addition above it? The answer: install above it six steel trusses—each weighing 28 tons and spanning 78 ft. Then hang the rest of the addition on them!

The 61,000-sq ft addition being built above The Lankenau Hospital's one-story outpatient center is adjacent to its main five-story building. The outpatient center was originally designed for vertical expansion over an existing two-story projection simply by inserting some new columns.

However, when planning for the expansion began, critical operations housed in the projection could not be disturbed, and the space could not be relinquished for construction. The architect and engineer had to design a structural system that could be built without closing or disturbing these areas.

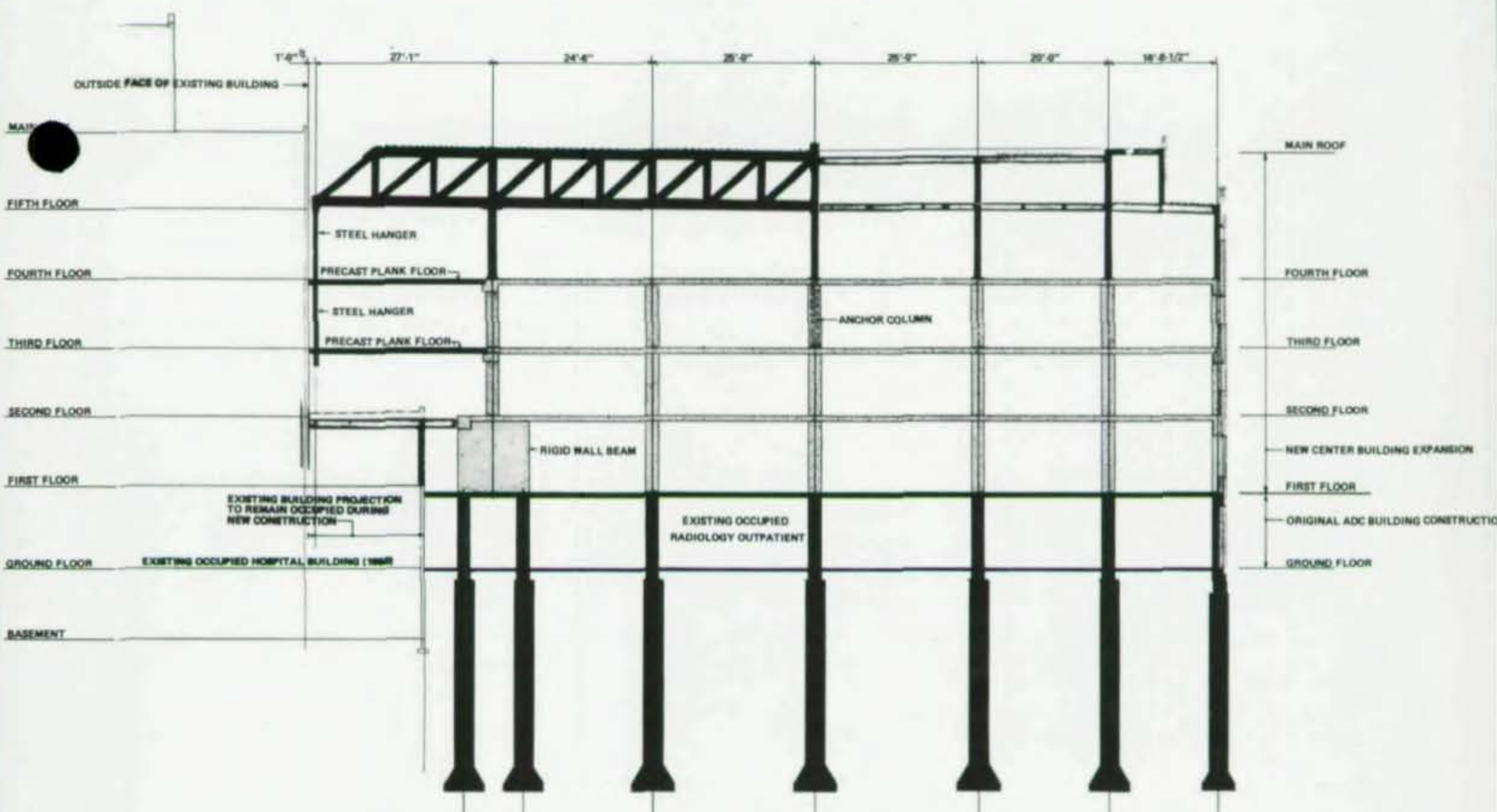
The structural system was designed to use existing foundations under the outpatient center. Since the top surgical floor



could accommodate a greater floor-to-floor height, six 78-ft long x 9-ft deep trusses—constructed of W14 sections—cantilever 27 ft on one end to support the three new floors hanging over the existing two-story projection and accommodate mechanical systems through the trusses. Since hangers support the precast plank floor, and only existing caissons are used, the hospital continued to use space in the projection during construction.

A 10-ft long rigid wall beam supports trusses and suspended floors by transferring loads to two smaller existing caissons. A composite steel column serves as an anchor column to support the trusses and the flat-slab floor on the fourth level. This flat slab construction provides enough counterweight on the steel-studded column for the vertical uplift force needed under full design load conditions. Precast plank flooring in the cantilevered area facilitates construction of self-supporting material where shoring from the lower floor was not possible. This struc-

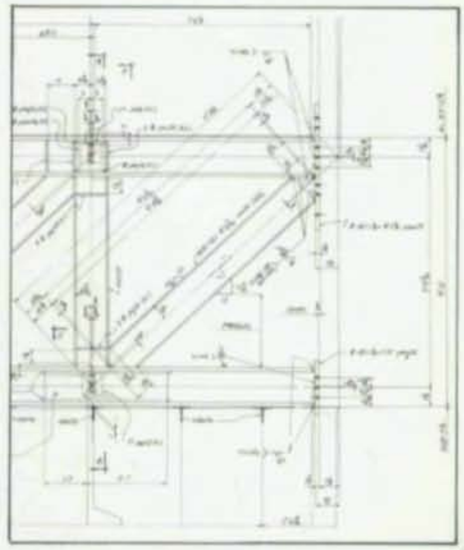
Giant trusses swing into place over existing hospital. Cantilever required loading each end at same time to avoid imbalance.



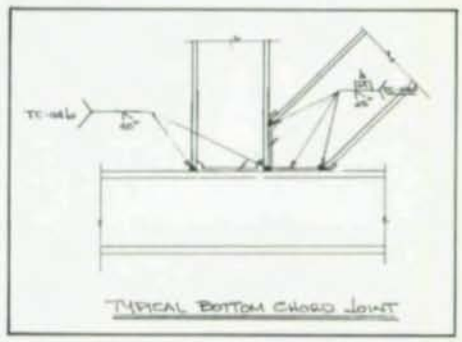
Suburban Philadelphia's Lankenau Hospital (l.) went "up" over rooms in operation. Outline shows new addition; detail (above) on structural building section.

tural system uses steel, concrete and precast to their fullest potential.

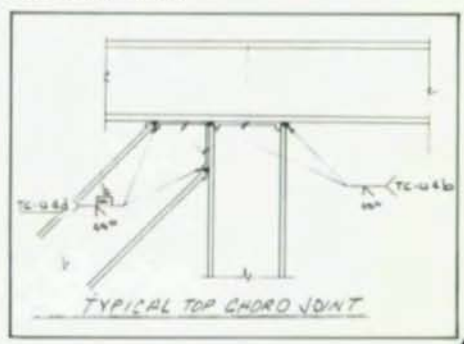
The addition, which will house radiology, clinical laboratories, operating rooms and administrative services, including medical records, is expected to be completed in the fall. This unique hospital project exemplifies the integration of architectural and engineering disciplines to meet the challenges of projects in functional—and economical—ways. □



Truss elevation



Typical bottom chord joint (above), and top chord joint (below)



Architect/Engineer
Ewing Cole Cherry Parsky
Philadelphia, Pennsylvania

General Contractor
John S. McQuade Co.
Philadelphia, Pennsylvania

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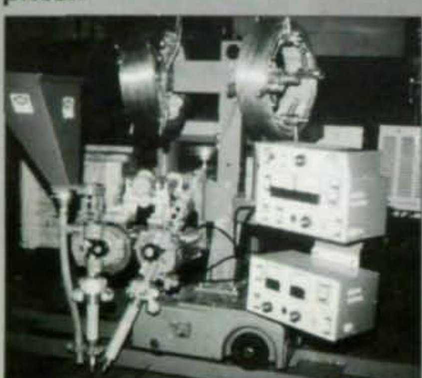

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Chicago Mercantile Exchange: Steel was Option on the Trading Floor

by Robert B. Johnson

Robert B. Johnson is a project engineer with Alfred Benesch & Company, a consulting engineering firm in Chicago, Illinois. He is the project engineer on the CME Center.

In early 1980, after years of phenomenal growth, the Chicago Mercantile Exchange (CME) considered the possibility of moving into larger quarters. The CME had outgrown its current quarters, built expressly for them, in just eight years. JMB and Metropolitan Structures, developer/contractor, purchased a nearby site in 1979 and were preparing preliminary plans for another type structure when they became aware of CME's desire for a new building.

Development studies for the site were revised in 1980 to incorporate CME's requirements for a major trading complex. Those requirements dictated several floors of office space and two column-free floors, one of which had to be about 40,000 sq ft. The final architectural solution featured twin 44-story office towers separated by a low-rise structure to house the twin trading floors. To achieve the desired floor area, the towers had to be cantilevered over the low-rise structure (Fig. 1).

In late 1980, preliminary design began in earnest after the CME committed themselves to moving into the new building. Several schemes were studied for the low-rise structure, nicknamed "The Box" by the architect/engineer. Large, post-tensioned girders were originally considered for support of the upper trading hall floor—a 180-ft span. These were immediately rejected by the architects/owner as not functional. The large, deep girders would result in compartmentalization of the interstitial space below the upper trading hall floor within the depth of the girders.

Space below the upper trading hall floor had to function as a huge mechanical room, and CME's program of providing mechanical/electrical service to the floor above dictated easy accessibility. Steel construction, using large trusses, afforded the possibility of spanning 180 ft and permitting maintenance men access through panels of the trusses. In addition, the exchange's communication system is extremely complex and fluid, with telephone lines in constant change. Therefore, a framing scheme was required which could accept continual changes in the mechanical/electrical systems.

During the preliminary design phase, a number of steel erectors were consulted by

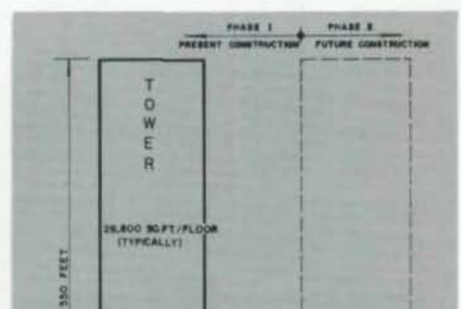


Figure 1—Elevation

Huge roof truss for "The Box" swings into place over CME trading floor between two towers (top photo).

Photos courtesy Robert B. Johnson

the architect/engineer/general contractor team to examine various alternatives to erecting the mammoth trusses. Any design element which could be engineered to aid and expedite construction procedures for the truss erection was reviewed. One major item which came to the attention of the engineer was the necessity to design parts of the upper trading hall floor for the heavy construction loads which might be applied during erection of its roof.

Another item critical to the design was the general contractor's wish that the tower and low-rise construction proceed independently. In addition, he needed to have the trading hall trusses erected as soon as possible to meet projected schedules of completion.

After reviewing all the input of the various parties—architect-engineer, general contrator-steel erector—the final design was begun in early 1981. The structural system called for a truss 175 ft long and 14 ft deep, weighing nearly 65 tons. The top chord is pitched slightly as a construction requirement to fit the truss under part of the seventh floor. All material for the trusses is 50 ksi. Top and bottom chords use jumbo column sections. Using the jumbo sections in tension necessitated that splices be bolted, rather than welded. The truss for the roof of "The Box," 150 ft long and 9½ ft deep, weighs about 35 tons (Fig. 2).

In September 1982, structural steel for the trading hall was erected. The six huge trusses, which support the upper trading hall floor, were barged to the site via the Ohio, Mississippi, Illinois and Chicago River waterways from the fabrication plant in Ambridge, Pa. A barge-crane, previously used to hoist structural steel for the new Columbus Drive bascule bridge in Chicago, was floated down the river to the site. The waterborne crane was chosen because the six trusses could not be manipulated either through or around Chicago viaducts. Transporting and erecting the trusses from the Chicago River proved the best solution to the problem.

First step was to lift the most westerly truss into its final position. Floor beams were then installed to tie the truss and the exterior wall together and provide lateral bracing for both elements. Next, two of the

Water-borne crane lifts 175-ft, 65-ton truss into position.

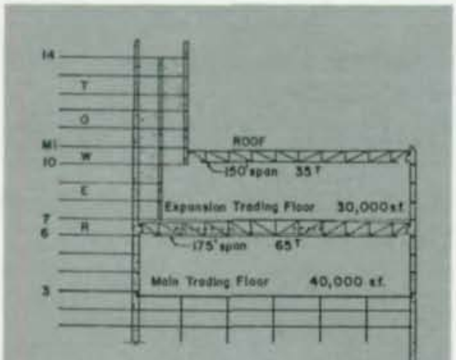
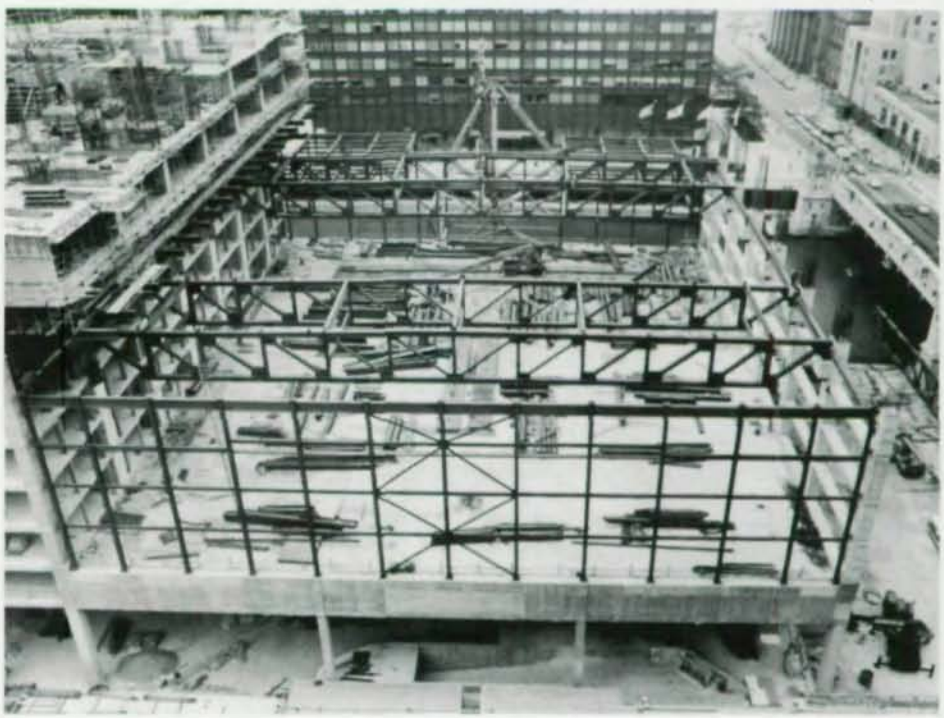


Figure 2—Section through "The Box"

Figure 3—"The Box" under construction

Figure 4—Trusses skid into final position



trusses, one at a time, were hoisted onto skid beams and braced together 30 ft apart (Fig. 3). The two trusses (130 tons) were then skidded across to the eastern end of the building into their final position (Fig. 4). Similarly, two more trusses were lifted and positioned. Finally, the last (sixth) truss was lifted by the crane into its final position.

This whole procedure of erecting the six trusses took only one week! And in the following weeks, the floor beams, metal deck and slab were installed.

In March 1983, the same procedure was repeated in a similar fashion for the roof. Roof trusses were fabricated in Indiana and barged to the site. However, in this instance the trusses were delivered in three separate pieces and field-bolted. Erection procedure for the six trusses was the same as before. Once again it took about one week for American Bridge Division to install the trusses.

The two trading halls—nicknamed the Supermarket II by its tenant, the Chicago Mercantile Exchange—have 40,000 sq ft and 30,000 sq ft of column-free space, respectively. The main hall will be the largest column-free trading hall in the world. Completion of this complex tower/trading halls is expected in early 1984. □

Typical scene soon to be enacted on new CME trading floors.

Architects
Fujikawa Johnson & Associates; and Space/Management Programs, Inc. (interior for CME)
Chicago, Illinois

Structural Engineer
Alfred Benesch & Company
Chicago, Illinois

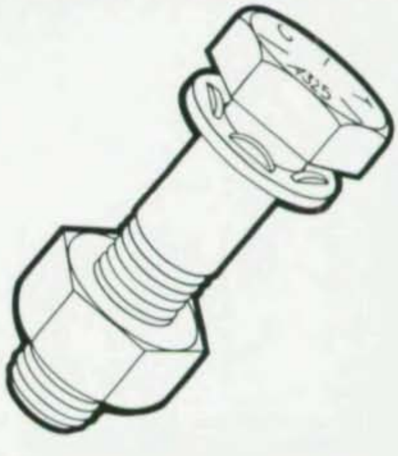
General Contractor
Metropolitan Structures
Chicago, Illinois

Steel Fabricators
USS Fabrication (trading floor)
Ambridge, Pennsylvania
and
Munster Steel Company, Inc. (roof)
Munster, Indiana

Steel Erector
American Bridge Division, U.S. Steel Corp.
Coraopolis, Pennsylvania

Owners
JMB Realty Corporation; and
Metropolitan Structures
Chicago, Illinois

Using Coronet Load Indicators saves over $75,000 on PPG Building.



Comparing the bolt inspection costs of two buildings recently erected in Pittsburgh, Pennsylvania, shows how using Coronet Load Indicators can mean significant cost savings.

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PPG Building, Pittsburgh, Pennsylvania. *Owner: Pittsburgh Plate Glass Co. Architect: Johnson/Burgee. Engineer: Skilling, Helle, Christiansen, Robertson. Fabricator: U.S.S. Fabrication, Division of United States Steel Corporation. Erector: American Bridge, Division of United States Steel Corporation.*

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Central Church: A Circle of Steel

Central Church is located on a 34-acre site in Memphis, Tenn., one ideally suited to the circular parking areas and vehicular circulation concept. The site master planning provides for playing fields, gymnasium, recreation center and senior citizen housing. The new $5-million facility includes a 5,000-seat sanctuary, offices, a 150-seat choir loft, baptistry, orchestra, kitchen, nursery, fellowship hall and 35 classrooms for a total of 105,200 sq ft. Its four-story circular design insures efficient internal circulation, as well as convenient accessibility to surrounding parking areas. The facility's design concept provides for continuing growth through a carefully phased construction program.

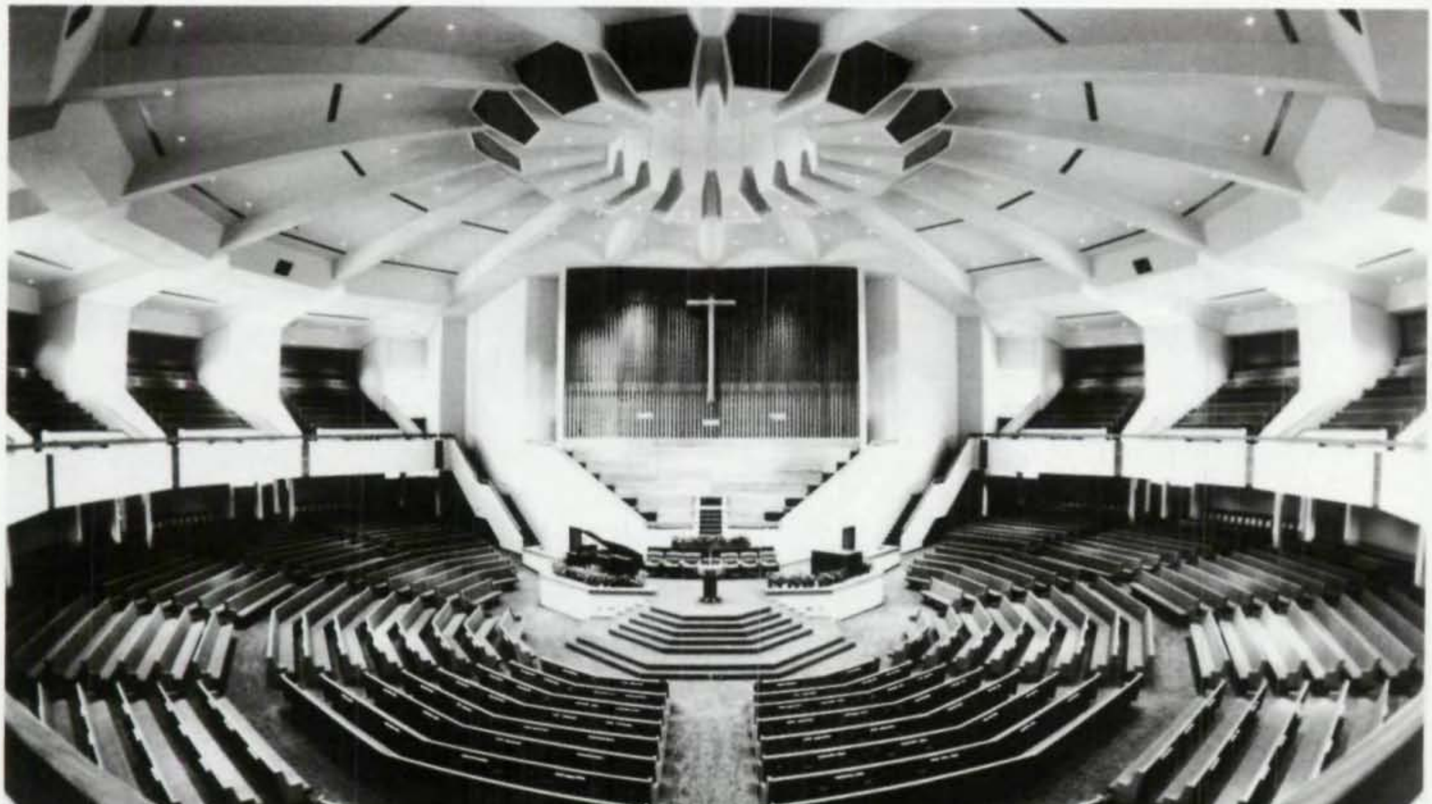
Design Criteria

The structural design criteria required the engineer to develop a clear-span roof structure for the sanctuary which would be compatible with, and enhance the aesthetics of, the building. To create a religious atmosphere, and retain a feeling of human scale, seating in the column-free sanctuary is focused on a centrally located pulpit. The ceiling slopes up toward the center of the sanctuary, in combination with a skylight located above the pulpit. These design considerations provide natural light and enhance aesthetic effects. In addition, the owner wanted a 90-ft cross on the center of the rooftop. Criteria also included the structural design of adjoining classrooms, offices, malls and other related spaces, plus provisions for future expansion.

Engineering Solutions

The engineering solutions meet the owner's requirements in a structure with these characteristics:

1. A 197-ft clear-span structure over a circular sanctuary constructed of 16 radially arranged trusses which vary in depth from 7 ft-6 in. at the eave to 30 ft at the peak. Both top and bottom chords of the trusses slope up toward the center of the sanctuary, which creates a structure resembling an "inverted morning glory." Clear height at the center of the sanctuary is 56 ft.
2. Due to limited headroom over seats on the upper level, no exterior tension ring could be provided to resist the outward thrust created by the upward sloping bottom chord. Therefore, the structure was designed as a totally self-stabilized, freestanding structure, with expansion joints at one end of each truss assembly to eliminate outward thrust on the supporting structure.
3. A 30-ft diameter skylight was required at the center of the sanctuary structure. The engineer used tension and compression rings at the bottom and top chords of the trusses, respectively. Above the top chord, the skylight opening extends upward into a 25-ft high cupola which supports the 90-ft cross.
4. Vertical X-bracing was an inevitable necessity for the structural framing system. However, locations of the X-bracing coincided with the location of fixed stained glass windows. To satisfy structural and aesthetic requirements, a specially designed connector resembling a cross was provided. This satisfied struc-

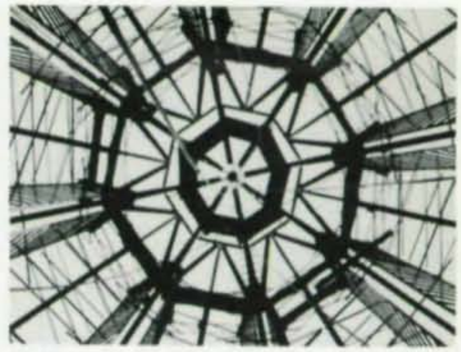


tural requirements, and is aesthetically pleasing when viewed through the stained glass windows.

5. Two layers of standard steel bar joists were employed: one at the top chord to support the roof system, the other at the bottom chord to support a wallboard ceiling. The ceiling, sculpted to reflect the structure above, produced an elegant cost-effective solution.
6. The two layers of bar joists create a large attic space in which catwalks permit service access to ceiling lights, audio equipment, HVAC and other mechanical equipment.
7. The roof system was extended radially outward from the sanctuary, with more conventional framing used to encompass various two- to four-story classrooms, office spaces and 30-ft mall areas.
8. The floor was a conventional composite construction, which has proven to be both sturdy and cost-effective.

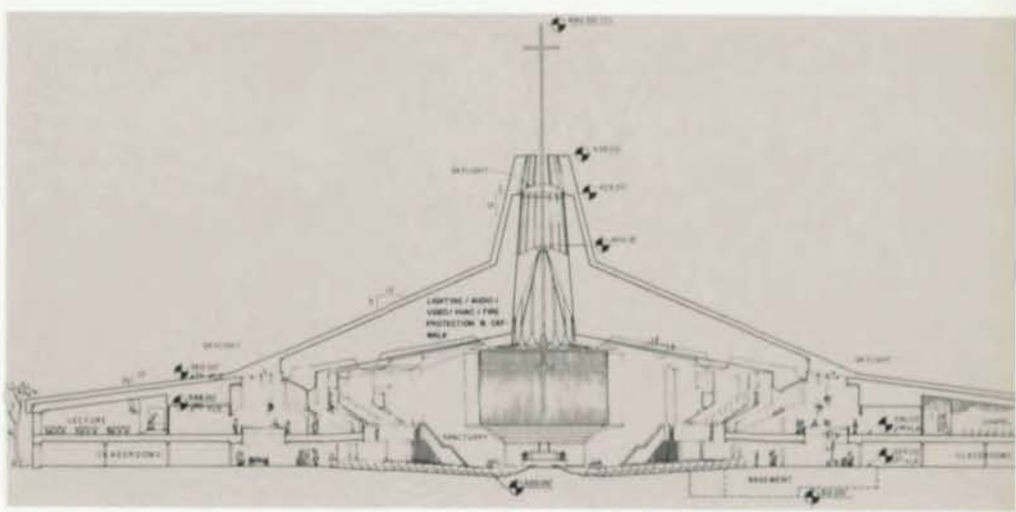
The project is a good illustration of the use of state-of-the-art design and construction techniques. In addition to its magnitude, the roof structure over the sanctuary is unique in itself. The steel skeleton provides an excellent backbone for aesthetically appealing, elegant, sculpture-type ceiling and roof finishes to emphasize the underlying structural elements. A soaring cross and the 30-ft open skylight in the center of the huge roof express the designer's boldness and imagination. Because of the magnitude and shape of the structure, steel was the logical material selection.

The structural engineer received the 1982 Engineering Excellence Award from the Consulting Engineers Council of Georgia for its state-of-the-art structural design. Also, Central Church was named "Outstanding Engineering Project, 1980-1981" by the Georgia Society of Professional Engineers. □



Central skylight and cupola (above and top r.) employ tension and compression rings at bottom and top chords of trusses.

Structural steel roof trusses radiate from central core to create spectacular clear-span space.



Magnificent 5,000 seat sanctuary (l.) of Central Church, Memphis, Tenn. reflects structural design criteria in developing clear-span roof structure. Cross section (r.) shows structural details and building's many functions.

Architect
IPG, Inc.
Valdosta, Georgia

Structural Engineer
Kun-young Chiu & Associates, Inc.
Valdosta, Georgia

Construction Manager
TMA, Inc.
Valdosta, Georgia

General Contractor
Martin Cole Dando & Robertson, Inc.
Memphis, Tennessee

Steel Fabricator
Pidgeon-Thomas Iron Company
Memphis, Tennessee

Owner
Central Church
Memphis, Tennessee

Barnes Building: Recycling an Illustrious Heritage

Not too many years ago, no one would have considered trying to save a 70-year old wool warehouse and convert it to a mixed-use office building. However, today's awareness of the value of old buildings as a resource, and not a liability, provides the architect and society an opportunity to redevelop and recycle them into a viable, desirable real estate investment. To have considered recycling a building that is close to a mile from the heart of Boston, and in an area that may still be years away from development, is equally amazing. However, the U.S. Government made a commitment in 1975 to completely renovate the Fargo Building—the same building many Americans may remember "visiting" before they joined the World War II and Korean conflicts.

Building Renamed—Now the Barnes Building

The Fargo Building is just one of many examples of how New England put one of its most valuable resources to work. The area is rich in history, and the fabric of that history is closely woven into its older buildings, many of which housed the American Industrial Revolution. These same older industrial buildings continue to serve as one of New England's greatest resources, often providing much needed, low-cost space for American entreprenuers. The old Fargo Building has quite an illustrious heritage, and has played a vital role in off-waterfront area industrial development, more recently as a government structure. The nine-story, 500,000-sq ft building occupies a full city block on Summer Street. Its original brick facade reinforced the architectural character created by the many brick warehouses located in that area. Now remodelled into a modern office building, and renamed the Barnes Building, the structure has a totally new image—one expected to serve as a positive influence on the quality and direction that might be taken in upgrading other buildings in the area.

Value Engineering Paid Off

In today's competitive environment, there is a great need for close cooperation between structural engineer, contractor, steel fabricator and erector to find the most economical solution in time and money to frame a building. While this is true to some extent on most buildings, it is a critical factor in reuse, rehabilitation and remodelling projects where an inappropriate design decision can seriously delay or financially jeopardize the project. This necessary cooperation takes many forms on private projects including design build, fast-tracking, and construction managers monitoring the design process to produce guaranteed maximum price based on partially completed structural drawings. While these strategies are applicable and widely used in the private sector, most of them are not applicable to competitively bid, fixed-price public contracts, where frequently the design engineer does not fully participate in the construction phase of the project.

To overcome some of these problems, some government contracts include a Value Engineering clause which encourages the contractor to submit changes in the design which could result in savings of time and money. The construction contract for the remodelling of the Fargo Building is unusual in that the Value Engineering Proposal included a major change in the seismic bracing systems and the supports for the exterior walls. Because of the size and complexity of the building and the tight construction

Existing Fargo Building circa 1909 (above) before reconstruction as Barnes Building (l.)

schedule, the proposal to change exterior framing was not for the faint-hearted.

The Fargo Building was originally framed with built-up structural steel columns and rolled beams arranged on 13 ft-5 in. x 15-ft bays. The floor system was flat tile arches with a concrete topping. The building was originally subdivided by a number of brick party walls extending from the basement through the roof. The exterior of the building was sheathed with conventional brick walls, partially enclosing the exterior columns and spandrel beams. The building, located on filled land, was supported by untreated wood friction piles. To accommodate the various uses over the years, portions of the interior transverse walls were removed and some local strengthening members installed on the floors. However, very little was done on the exterior elevations, which were 480 ft long, 120 ft wide and 124 ft high. These masonry walls were built with no control joints, and eventually cracks occurred which led to unsightly cracking and serious corrosion of much of the steel embedded in the walls at the upper floors.

When the Dept. of Defense decided to remodel the building to house 16 federal agencies, they decided to strengthen the floors, replace all exterior masonry with lightweight panels and strengthen the frame to conform to modern wind and seismic loading. The original structural scheme called for the installation of horizontal steel trusses under the floors, which delivered the lateral loads to a series of transverse braced steel frames. On the building exterior, lateral loads were to be resisted by a heavily reinforced ductile concrete moment-resisting frame, to be constructed around existing columns and spandrel beams. This frame, which involved 2-ft x 2-ft columns and 2-ft x 4-ft spandrel beams, would also serve as a support for the new curtain wall.

The $21-million contract to carry out the renovations was awarded in September 1978. However, after the general contractor analyzed the construction schedule it became clear that cast-in-place concrete framing around the perimeter of the building at all floors would seriously delay completion of the project. The contractor approached consulting engineers Brown, Rona Inc. to see what could be done to redesign the exterior wall and associated concrete framing under the Value Engineering clauses in the contract.

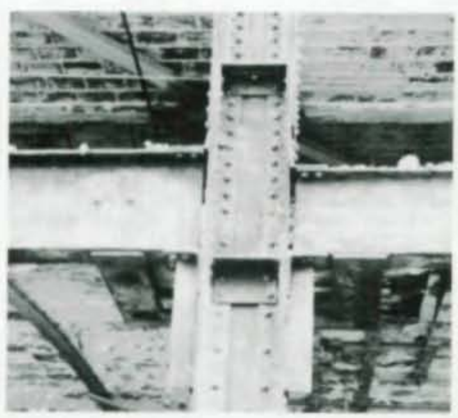
Switch to Steel

Discussions between Brown, Rona and the Corps of Engineers led to the idea of changing the concrete frame to steel. This would reduce the weight of the building and speed up construction. The weight reduction was important because of the largely unknown condition of the foundation system, along with a concurrent reduction in seismic forces. Substituting steel would also result in an exterior framing system structurally consistent with the horizontal steel trusses under the floors and the transverse steel-braced frames.

A preliminary design indicated that approximately 600 tons of structural steel would be required to resist lateral loads and support exterior walls. The contractor secured preliminary fabrication and erection prices, and schedules which indicated the concept was financially feasible, and would save considerable time because it permitted off-site fabrication. Most importantly, the speed of erection would permit demolition of larger areas of the existing brick facade of the building. The demolition scheme was critical to the job since the exterior masonry walls provided what little longitudinal lateral bracing there was in the building. Coordination between demolition and erection of the new framing became even more important when it was discovered the building was as much as 4 in. out of plumb. The new frame, or some extensive temporary bracing system, would have to be installed to resist the forces caused by this lack of plumbness.

Extensive Computer Analysis

Although the approximate preliminary design was done by hand methods, the final analysis and design were done on the Staad III Computer Program, analyzing a frame which included 324 members. The longitudinal and transverse walls were analyzed for dead and live loads, temperature variations, the specified wind loads and the seismic loads prescribed by the Common-



Sequence of construction: (l. above) near end, new exterior moment frame; mid-section, existing steel frame after masonry wall removal; far end, new cladding over new steel frame. Note sequence (l.) of cladding installation is left to right, not bottom up.

Existing riveted built-up column spandrel detail (l., above); above, new all-welded column/spandrel.

wealth's Building Code. To facilitate detailing and fabrication, and to control drift on the building to $h/400$, columns and spandrel beams were selected exclusively from 18- and 21-in. wide-flange sections of 50 ksi steel.

Column-to-beam joints were made with shear tabs and A325 bolts to carry the shear and full penetration welding of the flanges. The joints were reinforced with stiffeners and web doublers where required on the columns. Connections to exterior frames and the horizontal and transverse steel trusses were made with special plates to accommodate the individual field conditions as well as a continuous concrete bond beam at the edge of the floor.

Contemporary Panel Treatments

One of the major problems which precipitated modernization of the structure was the fact that it was actually seven contiguous structures with no expansion joints. The masonry exterior had experienced so much movement over the years that water had penetrated the masonry, which resulted in severe corrosion of portions of the exterior structural steel frame. To correct this deficiency, yet maintain the fireproofed structural steel frame and columns, a building skin study which examined 16 different exterior finishes was made.

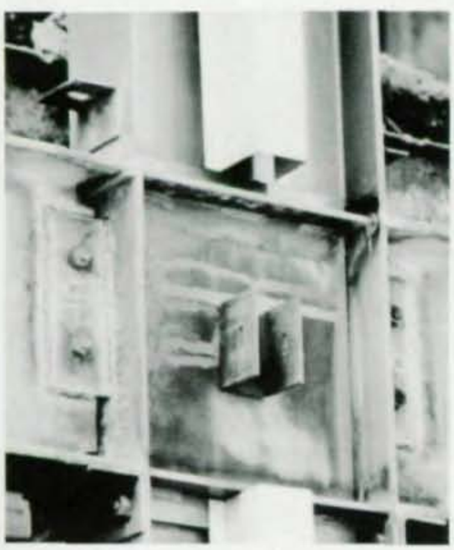
Early in the design process, the architect/engineer was encouraged by its client to consider giving the building a totally new image, one that would make it comparable to other new office structures in Boston. In recent years, the architectural community has been quite concerned with "contextualism"—how a building appears and fits into its surrounding environment. However, in this instance there was a greater overriding concern for creating a new image, while correcting the structural and architectural deficiencies. Since the existing brick had failed because of a lack of expansion and control joints, an all-important criteria was to replace the brick with a new system to correct this problem and yet provide greater thermal efficiency. In addition, the new skin would have to withstand up to 110 mph wind loading, as well as meet seismic design requirements.

Ultimately, a porcelain enamel steel panel system was selected because it not only met the performance criteria, but also provided a durable, long-lasting finish to withstand the assault of salt air and urban pollution. Aesthetically, the porcelain panel not only provided a much needed color palette, but also the practical and functional capability of being self-cleaning.

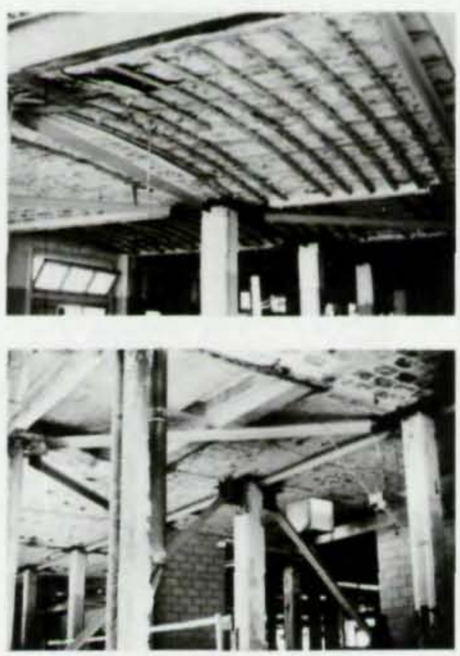
In the greater architectonic sense, the nearby Federal Reserve Bank Building served as the architectural antithesis to the new Barnes Building. The design of a strong, massive, horizontal structure leads the eye directly to the slender vertical statement at the end of Summer Street. By using a similar building color, the architect was able to visually unite these two very diverse architectural solutions, even though they are blocks apart. A dash of blue color was introduced, along with deeply recessed square-edge windows at the extremities of the structure, to accentuate and define the corners, over 480 ft apart on the Summer Street facade. The radius windows provide a rhythmic component which relieves monotony often found in office building facades. These radius edges with gasket frames also help soften the relationship between glass and metal panel, yet provide geometrical contrast to the square-edged louvers and end bay windows.

Thus, the urban aesthetics were married to the diversity of the area—and some very basic functional considerations were met at the same time. Removal and replacement of interior walls, coupled with the closely scheduled frame, permitted the skin to be installed in a highly efficient manner while providing protection to the new structural framing system. From the architect's point of view, this provided the ultimate synthesis in progressively uniting structural and architectural components.

The building was dedicated in April 1982. The successful execution of this imaginative Value Engineering Proposal which substituted structural steel for concrete saved the government \$270,000—and permitted beneficial occupancy of the new Barnes Building many months earlier than would have been possible with a concrete frame. □



Detail of new column/spandrel with existing spandrel behind



New horizontal diagonal bracing and existing barrel arch floors (top); and new horizontal and vertical diagonal bracing

Architect/ Engineer
Ganteaume & McMullen, Inc.
Boston, Massachusetts

Structural Engineer
Brown, Rona Inc.
Boston, Massachusetts

General Contractor
Wexler Construction Co.
Newton Highlands, Massachusetts

Steel Fabricator
Owens Steel Co.
Jacksonville, Florida

Owner
U.S. Army Corps of Engineers

Weathering Steel Bridges: Michigan Ban Sparks Multi-State Study

The State of Michigan imposed a statewide ban on the use of unpainted weathering steel on highway bridges in early 1980. That ban sparked a thorough and authoritative study of weathering steel on 49 bridges in seven states. The study was conducted by a task group of state and federal highway officials and steel company corrosion/metallurgical specialists. Findings of the study, under the auspices of AISI, should be of interest to states and localities who now have weathering steel bridges, as well as those who contemplate their construction.

The study was initiated after a statewide moratorium on using unpainted weathering steel in its highway program was declared by the Michigan DOT in March 1980. The bridge design engineer for the department stated the edict followed a limited moratorium on using the material in depressed roadways and in urban/industrial areas where heavy salting was prevalent. Both moratoriums came after a lengthy evaluation period which began with inspections of Detroit's 8-Mile Road Bridge. The inspections determined that corrosion rates were not tapering off, and that a probable cause was the confined environment which obstructed the wetting/drying cycles necessary for satisfactory performance of weathering steel.

Sharing Michigan's concern, the AISI organized the Task Group on Weathering Steel Bridges to study the problem. Members included state bridge engineers from Michigan, Illinois, Maryland, New York, North Carolina and Wisconsin; the chief engineer of the New Jersey Turnpike Authority; and representatives from the Federal Highway Administration and AISC. Steel company members include those from Armco, Bethlehem, Inland and U.S. Steel Corp.

Robert F. Wellner of Bethlehem Steel Corporation, chairman of the task group, noted: "It was not our purpose to advocate the use of weathering steel in highway bridges everywhere. Our objective was twofold: to determine if the Michigan bridge situation indicated a general problem, or one peculiar to that state; and to report all the findings, which will help states and specifying agencies to evaluate the practicality of using unpainted weathering steel in a particular bridge program."

The key task undertaken was inspection of existing weathering steel bridges in all states represented on the task group. These states had a total of 938 weathering steel bridges when inspections began. A uniform inspection form and procedure was developed, based on varying bridge site conditions—including amount of traffic, geometric features and exposure to de-icing salts. Each inspection team consisted of an industry corrosion engineer and representatives of the responsible owning agencies.

Study Findings

The data collected on the effects of long-term exposure of weathering steel found that of all the bridges inspected, 30% showed good performance in all areas, 58% showed good performance with moderate corrosion in some areas; 12% showed good overall performance with heavy corrosion in some areas.

Summarizing the inspection findings, one or more of the following four factors are believed responsible for formation of non-adherent, flaky rust: (1) water runoff, contaminated with de-icing salts during winter months, which drains through leaky seals and open joints or expansion dams; (2) water and de-icing salts leaking through cracks in the deck; (3) contaminated water runoff draining directly over the edge of the bridge onto the superstructure; (4) rust and dirt caused by tunnel-like conditions, which concentrate road sprays from the under-bridge traffic to result in the accumulation of water, dirt and possibly salt on the superstructure.

Howard S. Heydon, chief engineer of the New Jersey Turnpike Authority, states: "I am pleased we had the investigation, for it focused attention on the problem of salt water runoff and the degree of corrosion it can cause on A588 steel. We have learned to pay heed to its drainage.

"Unpainted weathering steel comprises 100,000 of the 300,000 tons of steel used on the Turnpike. We know it has paid for itself several times over by eliminating the need for initial and maintenance painting. This is also a big safety boost, for there is no need to close heavy traffic lanes to accommodate painting. We were able to cut costs further by using a thinner gauge of weathering steel, yet provide the equivalent strength of regular steel."

The study found de-icing salts to be the

Structure, which carries I-26 over Green River (N.C.) is 12 years old. One of earliest bridges of weathering steel, and for years the longest span, it is subject to light traffic and salt conditions.

major contributor to excessive corrosion of most bridge materials, and weathering steel is no exception. In areas where the steel is continuously exposed to de-icing salts, a flaky, non-adherent rust forms and the rate of corrosion does not diminish.

The task group communicated with the Salt Institute to determine the quantities of de-icing salt applied at locations surveyed. Of all, the Michigan structures are exposed to the most de-icing salts, especially in the Wayne County and Detroit areas. Also, as noted on the inspection reports, the rate of corrosion measured by the quantity of non-adherent rust was greater in areas where the steel was directly exposed to de-icing salts at leaky joints. And Michigan has introduced more deck joints in multi-span bridges than most other states, due to its widespread use of cantilevered/suspended spans.

Mill Scale. The oxide that forms on steel during cooling after hot rolling has little effect on the long-term performance of weathering steel. Where aesthetics are a consideration, the mill scale should be removed by sandblasting to promote the earlier development of a uniform, protective patina.

Corrosive Deposits. Analysis—by wet chemistry, spectrographic and X-ray diffraction techniques—of the flaky, non-adherent rust showed significant amounts of chlorides. The accumulation of rust deposits on horizontal steel surfaces further aggravates corrosion by providing a poultice or constantly wet environment.

Fatigue Life. No evidence of fatigue problems (premature fracturing) due to corrosion was observed on any of the bridges inspected. However, since the age of the structures ranged from only four to 16 years, this study cannot be considered conclusive in terms of this factor.

One of the study's findings was that portions of certain bridges were exposed to aggressive conditions requiring remedial painting. Since current field painting practices may be inadequate when the surface is contaminated with chlorides, the task group contacted the Steel Structures Painting Council to ask for assistance in solving this problem. The SSPC is now conducting a study on the cleaning and painting of weathering steel subjected to aggressive environments. When completed, the results will be made available.

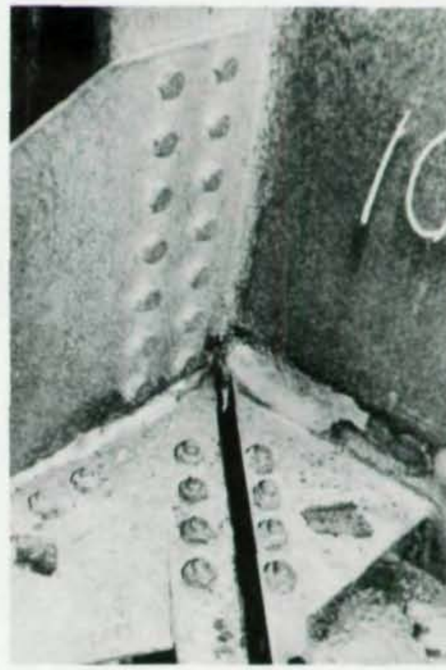
According to Edward V. Hourigan, director of Structure Division for N.Y. State DOT, "In New York, we have had no great problems in employing weathering steel on some 60 bridges throughout the state, and still denote it as the normal standard for bridges. Obviously, there are locations where we would not specify it, such as areas subject to constant wetness or excessive salting. We think the task group's report was complete and it should be helpful to those who contemplate the use of weathering steel."

"In its conclusions," says Robert Wellner, task force chairman, "the study finds that selection of corrosion-resistant steels for bridges is a matter of engineering judgement. Some of the factors to be evaluated are aesthetics, the safety resulting from no painting over traffic, savings derived from elimination of painting and the greater strength of weathering steel versus its higher initial cost.

"Most important of all," he states, "is environmental evaluation of the overall bridge site. Any conditions which create continuous wetting over a long period of time and/or chlorides on the steel have to be avoided. In Michigan, for example, local conditions include exceptionally heavy use of de-icing salts, and design details such as pin/hanger connections for cantilevered/suspended spans. Due to the potential for leakage at bridge joints, design and detailing play an important role in avoiding possible problems at critical points of a structure."

The investigation has not produced any evidence to warrant major changes in the decision-making process used to decide upon the specification of corrosion-resistant steels. The vast majority of such steels installed in this country perform in a satisfactory manner. □

Note: *a copy of the complete 32-pg. report, replete with 4-color photographs*—Performance of Weathering Steel in Highway Bridges—*is available from American Iron & Steel Institute, 1000 16th St. NW, Washington, D.C. 20036. We are endebted to AISI for permission to adapt this material.*



All steel on I-26 Green River Bridge is in excellent condition, except where subject to ponding of heavy runoff from leaky expansion joint. Fabric trough at open finger joint is perforated, concentrating runoff water.

Nine Mile Creek Bridge, Syracuse, N.Y. is in industrial environment, with heavy salting and chemically polluted stream below.

Seventeen Years . . .
and Still Building with Steel

The Hillier Group, Princeton, N.J., affirms a 17-year history of building with steel. In this period, the firm has designed over 15,000,000 sq ft of space—much of it steel-framed. Using steel framing has been an important part of their award-winning design approach. Steel has proven to be cost-and-time effective and has allowed innovative designs to be translated into actual buildings.

Bryant College was the first large-scale project undertaken by Hillier. Based on a time-critical need to relocate the campus from downtown Providence, R.I., in an elapsed time of 26 months, the college was moving to a 288-acre site in rural Smithfield. Beginning in 1967, Hillier designed a completely new campus to accommodate the student body of 2,500, plus future expansion of the college. The campus included dormitories and married student housing; an academic steel-framed "unistructure" with 280,000 sq ft of classrooms, faculty and administrative offices; a student activity space; a gymnasium/athletic complex; a president's residence; a sewage treatment plant; and complete development of the 288-acre site.

To meet the schedule imposed by the commitment of the owner to vacate existing facilities, a "fast-track" method was adopted. As a direct result of working with smaller contracts, which saved time, and also the methods of construction specified, the college realized a five percent savings in overall cost. The time-and-cost effectiveness of the steel infrastructure contributed to the success of the project—and permitted Bryant to move into a totally new campus in only 26 months, and under budget.

Award-Winning Home

Hillier used steel to frame his personal residence in 1970. Because of water conditions on the site, the house is elevated on a steel frame, and surrounded by a deck. Winner of the AISC Architectural Award of Excellence and the Homes for Better Living Award, the floor plan revolves around an ever-changing skylit atrium which floods the house with sunlight, to contrast with the dark woods around it.

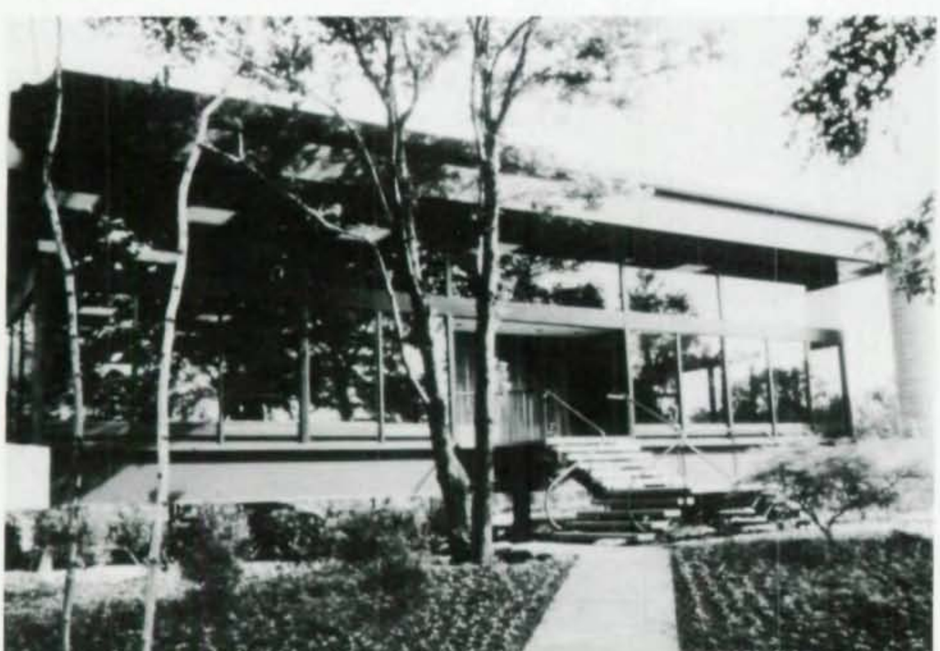
Made possible by the use of steel framing, design of Hillier's home proved so functional that the family has continued to live there even as it has grown and its needs have changed.

Another winner of an AISC Architectural Award was the first home office for The Hillier Group. Completed in 1973, it was the first building in the state of New Jersey to be constructed under the mandated requirements of the Flood Plain Act. The design solution permitted a building to be constructed in the flood plain—an otherwise

Bryant College, Smithfield, R.I.

Award-winning personal residence

Architect
The Hillier Group
Princeton, New Jersey



First Hillier Group home office

Second Hillier home office—also in steel

unusable site—and turned a swamp into a tax ratable site.

To meet the flood plain requirements, the building is "hung" from four major concrete-encased steel columns. It was constructed on an elevated steel frame with parking below the steel deck. Columns at the outer corners increase its visual size to a scale in keeping with the surrounding fields. The building is an open two-story high studio. Its floor space was increased by inserting mezzanine platforms of steel columns and decking several years after completion. Fifteen-foot trees within the building divide areas and bring the outdoors into the building.

A Renovated Hospital

The Hillier Group used steel in 1978 at Butler Hospital (N.J.) to frame a glass-enclosed garden to connect the old hospital with a new wing. Situated on 114 acres, the hospital had become obsolete. The 135-year old Gothic Revival psychiatric hospital needed to be renovated to bring it up to life-safety codes, to become functionally efficient, and to install state-of-the-art mechanical/electrical systems. In renovating the existing building and in constructing the new addition, the architect used steel.

Fifteen years after the firm's founding, Hillier moved his growing firm into its second home office building. Once again steel proved to be the most cost-and-time effective method to achieve the innovative design. With a sloped-roof designed for passive solar advantage, the reflective glass skin of the building is attached to drywall system supported by steel mullions. The building was featured in *Modern Steel Construction*, Fourth Quarter, 1982.

Largest Project to Date

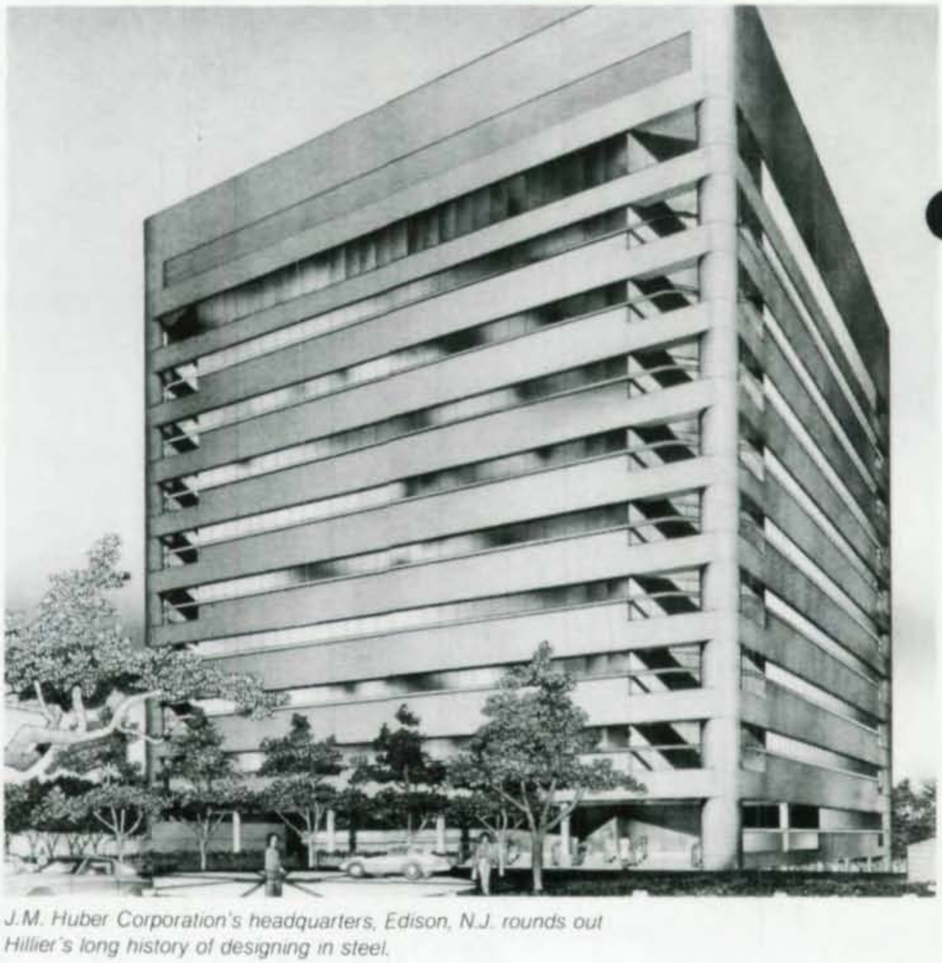
Beneficial Center—over 1,000,000 sq ft—the largest project designed by The Hillier Group to date, was dedicated in August 1982. The fast-track project took just four years from the time the architect was commissioned until it was completed. The old world, Flemish bond-brick exterior skin is attached to a steel infrastructure. If traditional brick construction had been used, the giant complex would have taken much longer to complete.

Beneficial wanted to humanize its corporate offices. To realize this concept, a corporate "village" of individual buildings was designed—each building houses a defined category of corporate and support activity. The buildings were then linked by arcades at the main plaza level and by skylit tunnels at parking levels below the complex. The focal point of the village is the 88-ft campanile, a clock tower concealing a water storage tank. Because of the project's visibility from surrounding hilltops, employee cars are concealed beneath the complex and in two independent brick-faced garages.

Continuing the architect's long history of building with steel is a 200,000-sq ft corporate headquarters of J.M. Huber Corporation, currently under construction in Edison, N.J. The building, constructed in an environmentally sensitive area, actually spans a brook which runs through the site. Concrete-encased steel columns support the elevated steel framing. Again, the strength and adaptability of steel permitted a building to be constructed on an otherwise unusable site.

In Hillier's 17-year history, using steel framing has permitted their innovative designs to become functional, trendsetting buildings. The Hillier Group depends on the effectiveness of steel for its strength, its lightness and its flexibility. □

Beneficial Center, Peapack-Gladstone, N.J.—firm's largest project to date.



J.M. Huber Corporation's headquarters, Edison, N.J. rounds out Hillier's long history of designing in steel.

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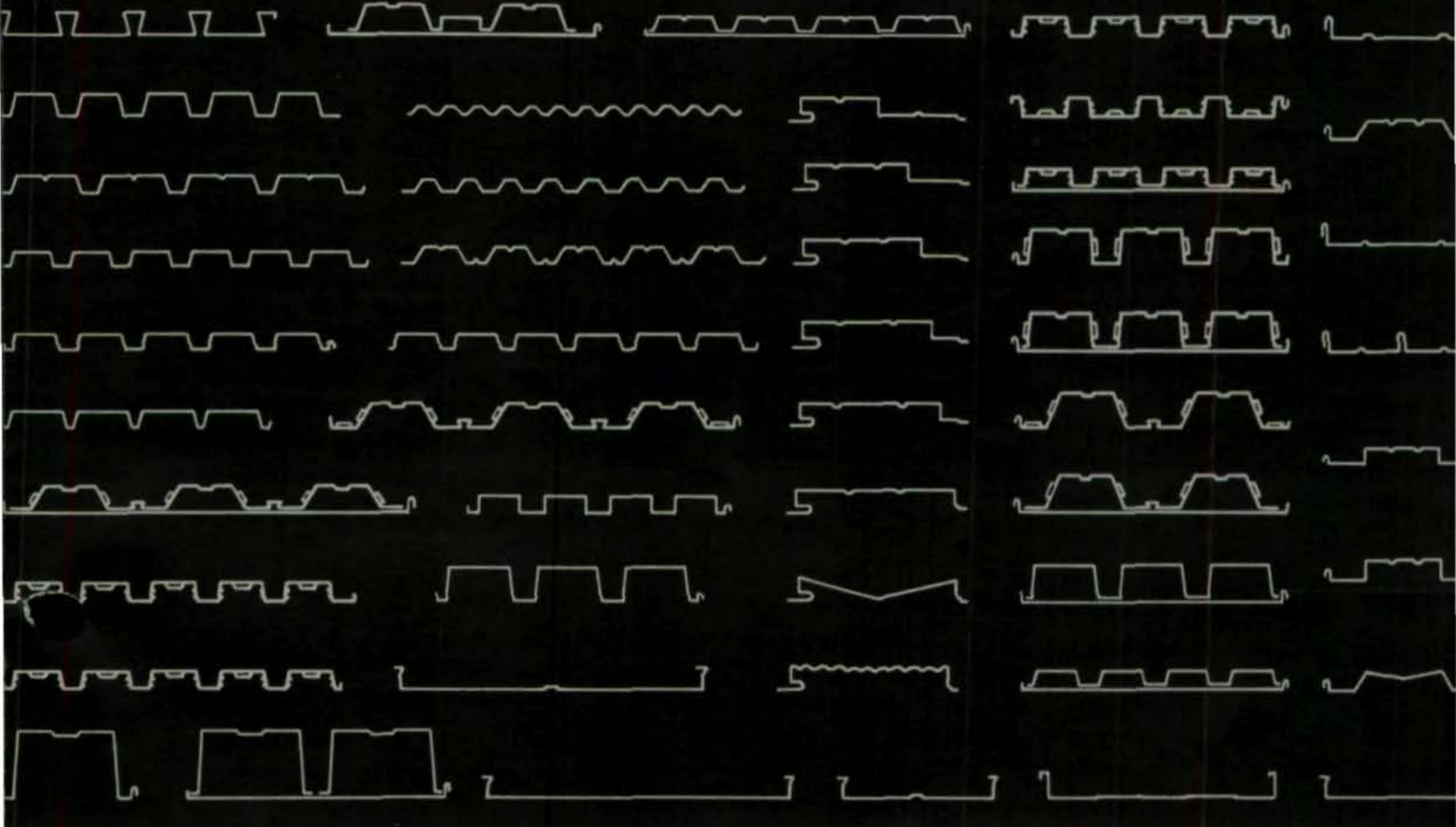

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