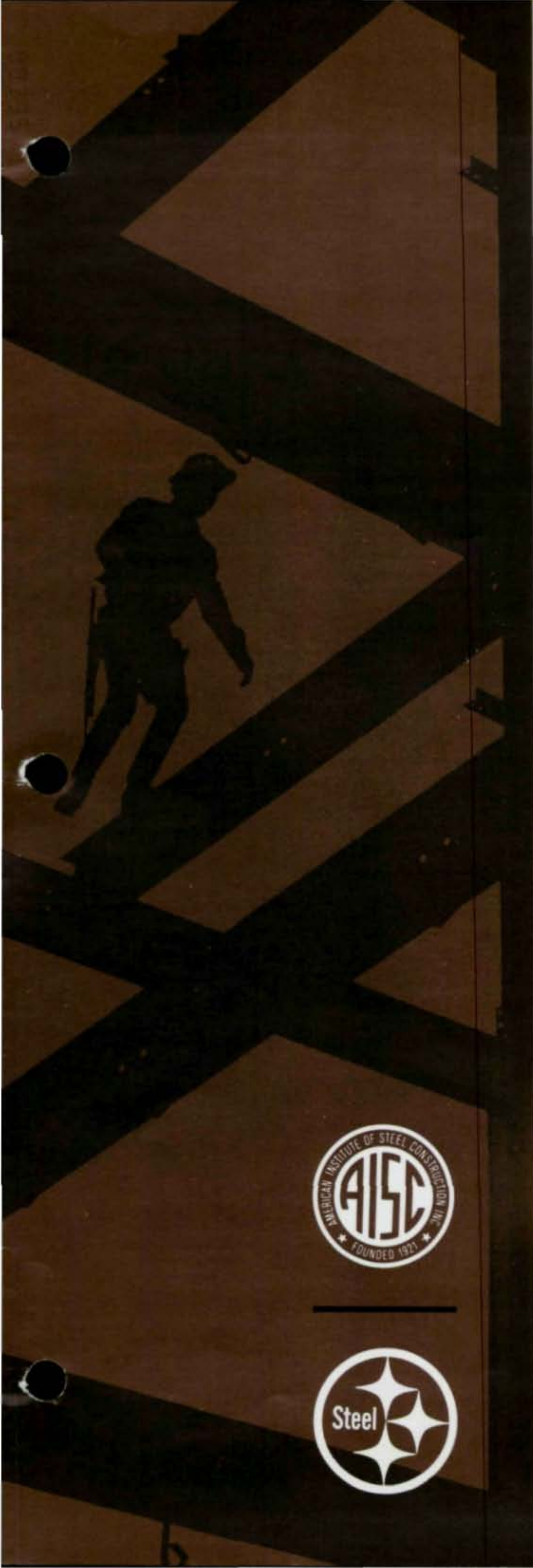


VOLUME XXIV NUMBER 2/SECOND QUARTER 1984

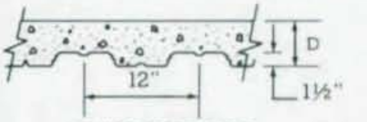
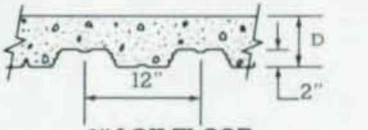
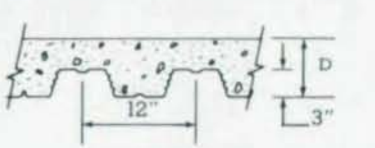
MODERN STEEL CONSTRUCTION

A Triple-Crowned Structural System
Steel Bridges the Gap to Tomorrow
Coast to Coast—Condos Go Steel
Steel-Framed Parking Structures in Up Trend
The Hills are Alive Again!
"Crossing Over" to a New Environment

DECK DESIGN DATA SHEET

NO. 4

COMPOSITE DECK/SLAB I VALUES

DECK/SLAB COMBINATION	TOTAL SLAB DEPTH 'D'	COMPOSITE I (Moment of Inertia),* inches⁴/ft. of width													
		150 PCF, n=9							115 PCF, n=14						
		Gage							Gage						
		22	21	20	19	18	17	16	22	21	20	19	18	17	16
 1½" B-LOK UNITED STEEL DECK, INC.	4.00"	3.5	3.6	3.8	4.1	4.4	4.7	5.0	2.8	2.9	3.1	3.3	3.6	3.8	4.0
	4.50"	4.9	5.2	5.4	5.8	6.2	6.6	7.0	3.9	4.1	4.3	4.7	5.0	5.4	5.7
	4.75"	5.8	6.1	6.3	6.8	7.3	7.8	8.2	4.6	4.8	5.1	5.5	5.9	6.2	6.6
	5.00"	6.8	7.1	7.4	7.9	8.5	9.0	9.5	5.4	5.6	5.9	6.4	6.8	7.2	7.6
	5.50"	9.0	9.4	9.8	10.5	11.2	11.9	12.5	7.1	7.4	7.8	8.4	9.0	9.5	10.1
	5.75"	10.3	10.8	11.2	12.0	12.8	13.6	14.3	8.1	8.5	8.8	9.5	10.2	10.8	11.4
	6.00"	11.8	12.3	12.7	13.6	14.5	15.4	16.2	9.2	9.6	10.0	10.8	11.5	12.2	12.9
 1½" LOK-FLOOR UNITED STEEL DECK, INC.	4.00"	3.5	3.7	3.9	4.2	4.5	4.8	5.1	2.8	3.0	3.1	3.4	3.6	3.9	4.1
	4.50"	5.0	5.2	5.5	5.9	6.3	6.7	7.1	4.0	4.2	4.4	4.7	5.1	5.4	5.7
	4.75"	5.8	6.1	6.4	6.9	7.3	7.8	8.2	4.6	4.9	5.1	5.5	5.9	6.3	6.6
	5.00"	6.8	7.1	7.4	7.9	8.5	9.0	9.5	5.3	5.6	5.9	6.4	6.8	7.2	7.6
	5.50"	9.0	9.4	9.8	10.5	11.1	11.8	12.4	7.0	7.4	7.7	8.3	8.9	9.4	10.0
	5.75"	10.2	10.7	11.1	11.9	12.6	13.4	14.1	8.0	8.4	8.8	9.4	10.0	10.7	11.3
	6.00"	11.6	12.2	12.6	13.5	14.3	15.1	15.9	9.0	9.5	9.9	10.6	11.3	12.1	12.7
 2" LOK-FLOOR UNITED STEEL DECK, INC.	4.50"	4.6	4.9	5.1	5.5	5.9	6.2	6.6	3.7	3.9	4.1	4.4	4.8	5.1	5.4
	5.00"	6.2	6.5	6.8	7.3	7.8	8.3	8.8	4.9	5.2	5.4	5.9	6.3	6.8	7.2
	5.25"	7.1	7.5	7.8	8.4	9.0	9.5	10.1	5.6	6.0	6.2	6.8	7.2	7.7	8.2
	5.50"	8.1	8.5	8.9	9.6	10.2	10.9	11.5	6.4	6.8	7.1	7.7	8.2	8.8	9.3
	6.00"	10.5	11.0	11.4	12.3	13.1	13.9	14.7	8.3	8.7	9.1	9.8	10.5	11.2	11.8
	6.25"	11.8	12.4	12.9	13.9	14.7	15.6	16.5	9.3	9.8	10.2	11.0	11.8	12.6	13.3
	6.50"	13.3	13.9	14.5	15.5	16.5	17.5	18.4	10.4	11.0	11.4	12.3	13.2	14.0	14.8
 3" LOK-FLOOR UNITED STEEL DECK, INC.	5.50"	7.8	8.2	8.6	9.2	9.8	10.5	11.1	6.2	6.6	6.9	7.4	8.0	8.5	9.0
	6.00"	9.8	10.3	10.7	11.5	12.3	13.1	13.9	7.8	8.2	8.6	9.3	10.0	10.7	11.3
	6.25"	10.9	11.5	11.9	12.9	13.7	14.6	15.5	8.6	9.1	9.6	10.4	11.1	11.9	12.6
	6.50"	12.1	12.7	13.3	14.3	15.2	16.2	17.2	9.6	10.1	10.6	11.5	12.3	13.2	13.9
	7.00"	14.9	15.6	16.3	17.5	18.6	19.9	21.0	11.8	12.4	13.0	14.1	15.0	16.1	17.0
	7.25"	16.4	17.3	18.0	19.3	20.6	21.9	23.1	13.0	13.7	14.3	15.5	16.6	17.7	18.7
	7.50"	18.1	19.0	19.8	21.3	22.6	24.1	25.4	14.3	15.1	15.7	17.0	18.2	19.4	20.6

*The 'I' values shown in the table have been calculated by using the transformed section method of analysis—concrete converted to equivalent steel. The averages of the cracked and the uncracked I values are shown. Use E=29,500 ksi (and the I shown) for live load deflection calculations.
All values are based on **UNITED STEEL DECK, INC.** deck sections.

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CONTENTS

1984 FELLOWSHIP AWARD WINNERS RECENTLY NAMED

The eight winners of AISC's 1984 Fellowship Awards competition have been named. Each student receives a $4,000 study fellowship and an additional $750 goes to their academic department heads to administer the awards. The students are judged by an outstanding award jury on the basis of their grade point averages, faculty recommendations—and the contributions their expected study programs make to the engineering professional and the structural steel industry as a whole.

The 1984 winners are:

Kevin V. Boldt, *University of Kansas*
David W. Hawkins, *Ohio State University*
Kurt R. Hoigard, *Illinois Institute of Technology*
William O. Kash, *University of Illinois, Champaign-Urbana*
Michelle Lutrell, *West Virginia University*
Michael D. Metcalf, *University of Kansas*
Todd S. Nottingham, *Montana State University*
Nancy M. Rizzuto, *Rutgers University*

JAMES M. FISHER AWARDED 1984 T.R. HIGGINS LECTURESHIP

James M. Fisher, Ph.D., P.E., Milwaukee, Wis., vice president of Computerized Structural Design, received AISC's prestigious T.R. Higgins Lectureship Award at The National Engineering Conference, Tampa, Fla. His winning lecture is, "Industrial Buildings—Guidelines and Criteria."

His award, an engraved citation and a check for $3,000, were presented at NEC by Robert P. Stupp, executive vice president of Stupp Bros. Bridge & Iron Company, St. Louis, Mo. Fisher will present the paper at five additional cities and events during the year.

1984 INTERNATIONAL ENGINEERING SYMPOSIUM SET

The 1985 International Engineering Symposium on Structural Steel is scheduled for May 22-24 at The Palmer House, Chicago. Co-sponsors are AISC and the Canadian Institute of Steel Construction. Combining their resources, the two associations will bring together from all over the world experts in structural engineering, materials, design, fabrication and erection. For 1985, the Symposium will supersede the National Engineering Conference sponsored annually by AISC. The complete program will be released within the next few weeks. Write Dept. of Public Affairs, AISC, 400 N. Michigan, Chicago, IL 60611.

Republic Bank Center:
A Triple-Crowned Structural System

by P.V. Banavalkar and T. Abyad


Republic Bank Center, Houston, Texas

Rising 57 stories above downtown Houston, Republic Bank Center covers an entire block bounded by Louisiana, Capitol, Smith and Rusk streets. The project—two interrelated entities, a banking hall and a tower (Fig. 1a)—contains 1.52-million sq ft of office and retail space.

Occupied by Republic Bank Houston, the banking hall encompasses the old two-story Western Union Building, which serves the telecommunications network of the southwest region. This major telecommunications center had to remain in operation during and after completion of the banking hall and adjoining tower. Excavation started in September, 1982 and nine months were required to carve out the 52-ft deep hole. The retention system consisted of concrete soldier piles and pre-tensioned tie-backs. Because of the construction of intersecting tie-backs at an interior corner of the Western Union Building, one side of the excavation next to the existing structure was retained by the strut and raker system. Under the footprint of the tower, an 8-ft thick mat with approximately 11,000 cubic yards of concrete was made in one continuous pour.

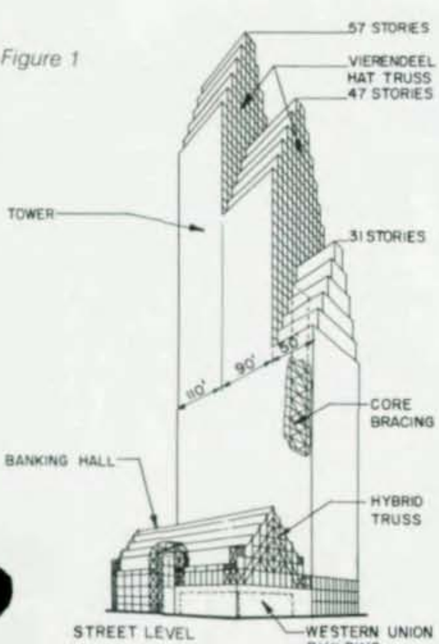
Triple-Crowned Structural System

Lateral Load Resistance: The tower wind-resisting system had to be tailored to fit the overall architectural composition of the building. Two major stepbacks along the longitudinal axis of the building created three adjacent units of varying heights.

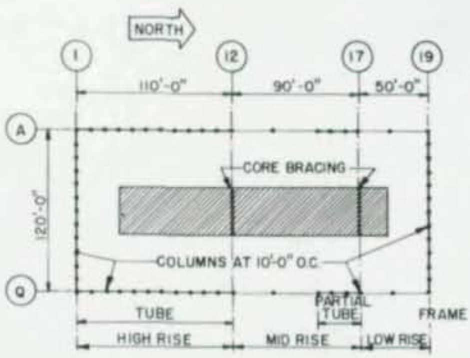
Varying stiffnesses had to be built into these three units to optimize the rigidities and minimize the overall twist of the building. Figure 1b shows, in plan, the balanced stiffness system used in the tower, a combination of three different structural systems assigned to three varying height

P.V. Banavalkar is executive vice president and chief structural engineer, and T. Abyad is senior associate, CBM Engineers, Inc., Houston, Texas.

Figure 1



A· STRUCTURAL SYSTEMS SCHEMATIC VIEW



B· STRUCTURAL SYSTEM REPUBLIC BANK TOWER units. This balanced structural system comprises a perimeter tube in the high-rise section, with one of its sides having a top Vierendeel outrigger hat truss over a braced interior bent (Fig. 3); a partial tube in the mid-rise section with an interior system of a Vierendeel outrigger truss similar to the one in the high-rise section; and a planar welded frame in the low-rise section.

As shown in Fig. 2a, the eccentricity between the applied lateral loads and the center of rigidity of the balanced structure is then kept to a minimum. Placement of the tube, the partial tube and the welded frame is also consistent with the required strength against overturning moment due to lateral loads, as seen in Fig. 2b, which shows the percentage of the overall overturning moment carried by each segment of the building system. Both Figs. 2a and 2b were prepared using the wind loads specified in Houston City Building Code. Column spacing for the exterior of the structure was consistent with the requirement of free spans for both the central arcade and the entry to the lower parking levels without unwarranted column transfers. The 10-ft spacing of the exterior column was also in tune with the architectural expression of the building.

Both all-steel and steel-concrete-composite exterior frames were considered as possible structural systems. The transfer and the stepback in the building profile with non-repetitive formwork above the stepback level made an all-steel structural frame more economical. Furthermore, initial analysis for the design of mat foundation indicated that increased weight due to the composite frame would mean having only three basement levels, rather than four in the banking hall south quadrant, because of the inadequate surcharge at the toe of the mat.

Floor construction is a conventional 4½-in. thick lightweight concrete slab supported on 2-in. deep metal deck. The column-free span of 42 ft between exterior columns and core columns is provided by steel composite beams supporting the metal deck.

Gravity Load Sequential Analysis: Like any other building, the exterior columns, which are part of the lateral load-resisting welded frame, do not carry equal floor loads. The frame action tends to equalize the load between these columns. However, for this structure, the linkage between exterior frame to interior core columns by means of outrigger Vierendeel hat trusses tends to redistribute the gravity load evenly between the exterior and interior columns. The redistribution capability of such frame and linkage between core and exterior columns depends on the change in stiffness characteristics as the building goes up, and the sequence in which gravity loads are applied (such as weight of steel structure, concrete floor, exterior skin of the building, mechanical systems, ceiling and partitions etc.). To account for this sequential change in stiffness characteristics of the structure, the gravity load analysis for the tower was performed in five segments (Fig. 3), with the applied gravity loads representing closely the construction schedule of the building. The sequential load analysis enabled the realistic prediction of load distribution in all the columns.

Innovative Structural Elements—the Flat Spandrel/Wide-Flange Column Tree Assembly

As shown in Fig. 4, the vertical granite

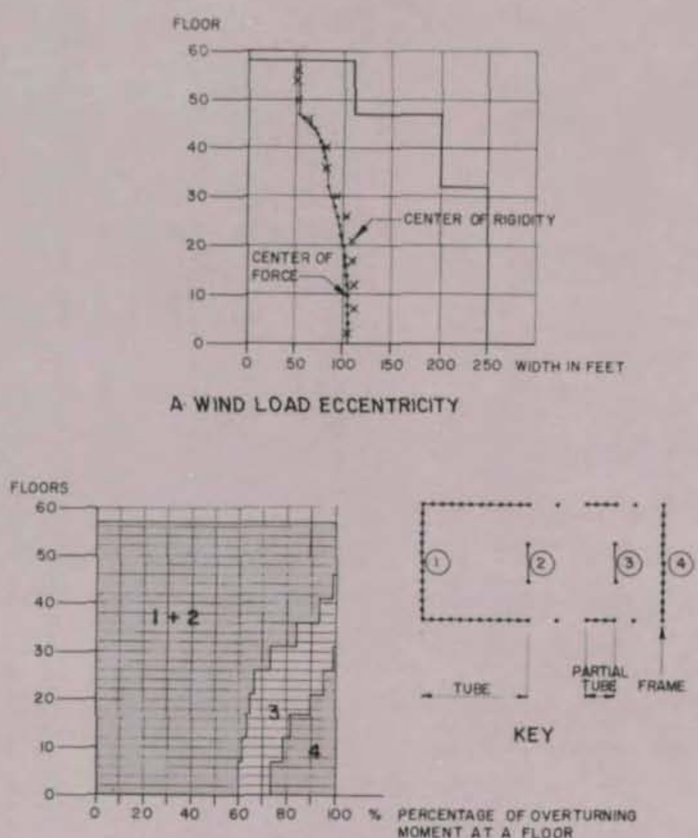


Figure 2

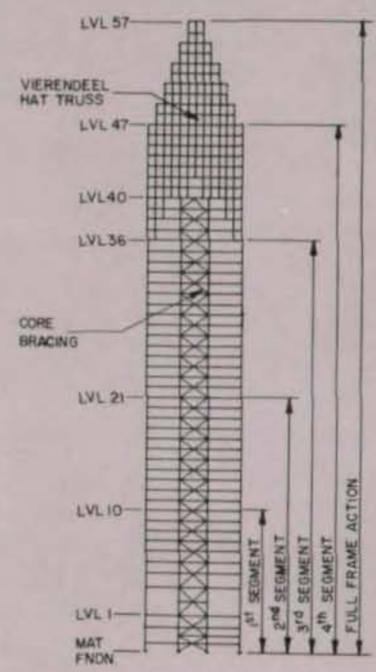


Figure 3

panels project beyond the plane of glass windows, thereby creating a multi-plane exterior surface for the building. Use of conventional wide-flange beam-and-column assembly would have resulted in location of the exterior columns 1 ft-8 in. from the exterior face of granite (Fig. 4b). Furthermore, it also would have required edge angles to form the slab edge. Therefore, to minimize the column projection into the lease space (see Fig. 4a), to reduce the projection of the granite skin from the base structure and eliminate the edge angle, the flat plate spandrel/wide flange column assembly was introduced for the first time ever. The new assembly was used as the main lateral-resisting element in the perimeter tubes and frames, with the wide-flange columns spaced at 10 ft o.c.

For the 10-ft spacing of columns in a welded frame, shear distortion of the spandrel contributes significantly to the lateral deflection of the structure, as opposed to the flexural distortion dominating conventional wide flange sections. To optimize the steel weight of the assembly, therefore, a 42½-in. deep channel-shaped flat-plate spandrel was used. The choice of optimized flat-plate spandrel reduced the spandrel weight between 25% and 30% compared to a conventional wide-flange section (Fig. 5). Due to the small angle (1°±) between the vertical axis and the principal axis of the spandrel, the biaxial bending stresses in the spandrel are insignificant. The top portion of the spandrel is braced to the floor slab by means of horizontal shear studs (Fig. 4a).

Figure 6 shows a typical tree column and spandrel field connection. The beams are spliced at the point of contraflexure (mid-span of the spandrel), which requires the transmission of vertical shear force only for the conventional symmetrical wide flange section. For the channel-shaped flat-plate spandrel, the splice should not only be adequate to transfer vertical shear, but also should be able to maintain the continuity of the shear flow. This was accomplished by connecting the flanges of the adjoining spandral with bolts.

The overall structural steel weight is 25.2 psf, which is quite efficient considering the building stepbacks and the 42-ft clear spans from core to building perimeter.

As opposed to symmetrically connected wide-flange beam-column assembly, the resultant bending stresses in the flanges of a flat plate spandrel do not pass through the shear center of a wide-flange column (Fig. 6c). The eccentricities of these forces, with respect to the shear center of column section, apply localized torques T and T′, as shown in Fig. 7. These torques generate torsional stresses in the column. The columns, therefore, were torsionally stiffened by exterior vertical 50-in. long stiffener plates at each floor level (Fig. 7). Using Vlassov's theory of open sections, the author developed a stiffness analysis method (to be published later) for the analysis of multi-span vertical plate stiffened columns. The additional warping stresses in the column flanges and the shearing stresses in the vertical stiffener plates were determined and accounted for in the design of the members and connections.

Serviceability Requirements

In order to determine a realistic amount of building drift and accelerations, a wind tunnel aeroelastic study was employed using a lumped four mass dynamic model with three degrees of freedom at each joint (Fig. 8). Results of these tests were used to determine the minimum acceptable required stiffness for the structure, both from

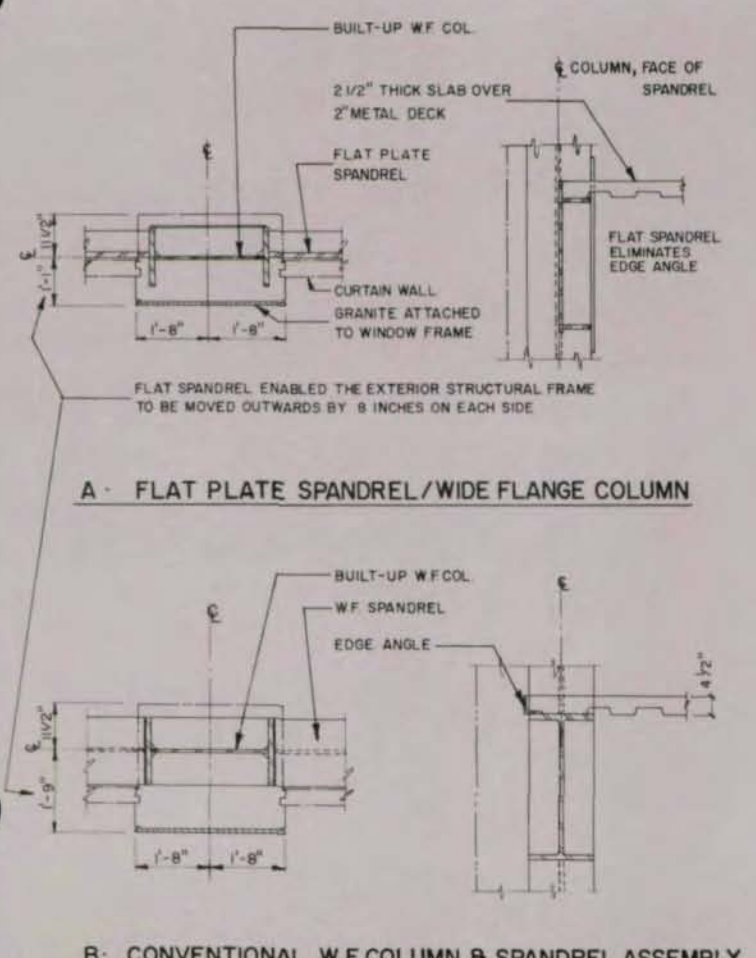


Figure 4

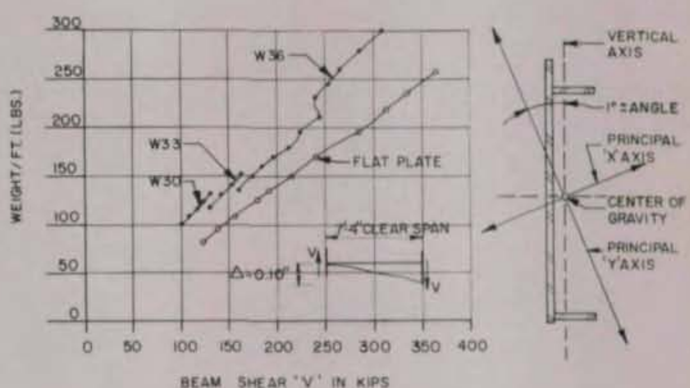


Fig. 5. Comparison between flat plate and conventional wide-flange beams

the standpoint of interstory drift and motion perception comfort of its occupants. A static model was also tested in the wind tunnel to determine the design cladding pressures.

Special Features

Batten Plated Columns and Arches: In the short axis of the building, at the transverse stepbacks, light composite batten plate column sections were used. These were fabricated from two C7 (lightweight) channel sections tied by intermediate batten plates. The overall geometry of these sections provided the necessary backup profile for the granite and curtain wall (Fig. 9). Also similar sections were used to form the curved arches at the gabled building roofs (Fig. 10).

Transfer at Loading Dock: Fig. 11 shows the interrupted tubular framing on one side of the building at the loading dock area. The interruption had to be compensated for by an overhead transfer truss and a diagonal brace to keep the flow of the forces constant on that side. The diagonal bracing field connections presented a challenging problem with forces upward of 2,500 kips to be transferred in a very short length of the members.

Mat Foundation and Column Elevation Adjustment: Because of the three varying height sections of the building, the mat foundation is subjected to unequal loads as opposed to a symmetrically loaded mat. This mat foundation not only dishes about both axes but also shows a tendency to rotate about the short axis of the structure. The four-level basement underneath the banking hall's south quadrant resulted in a reduced soil surcharge load at that corner of the mat. The reduced soil surcharge produced a twist rotation of the mat about its longitudinal axis.

Just as with any other multi-story building, there was differential axial shortening in the columns due to varying gravity load stresses. In order to provide level floors in the structure, vertical heights of the columns were adjusted taking into account the mat behavior and also differential gravity loads.

Banking Hall

The banking hall structure consists of two quadrants linked together at the location of the major roof arch (Figs. 12a and 12b). The south quadrant has the same number of basement levels as the main tower, except that its columns rest on spread footings. All basement levels and the ground floor are supported by the quadrant perimeter columns and additional interior columns terminating at ground floor level.

The wind resisting system consists of perimeter welded/braced frames (Fig. 12c) tied at the top with the roof space elements. The gable roof structure is a combination of stepped vertical trusses having a span of 110 ft together with stepped horizontal Vierendeel trusses which act as bracing elements to the vertical trusses. The highest point of the roof rises 132 ft above street level.

The north quadrant differs from the south in that it has only one usable floor immediately over the roof of the existing Western Union Building. The floor construction is 4.5-in. thick concrete slab on metal deck resting on 5-ft deep long-span

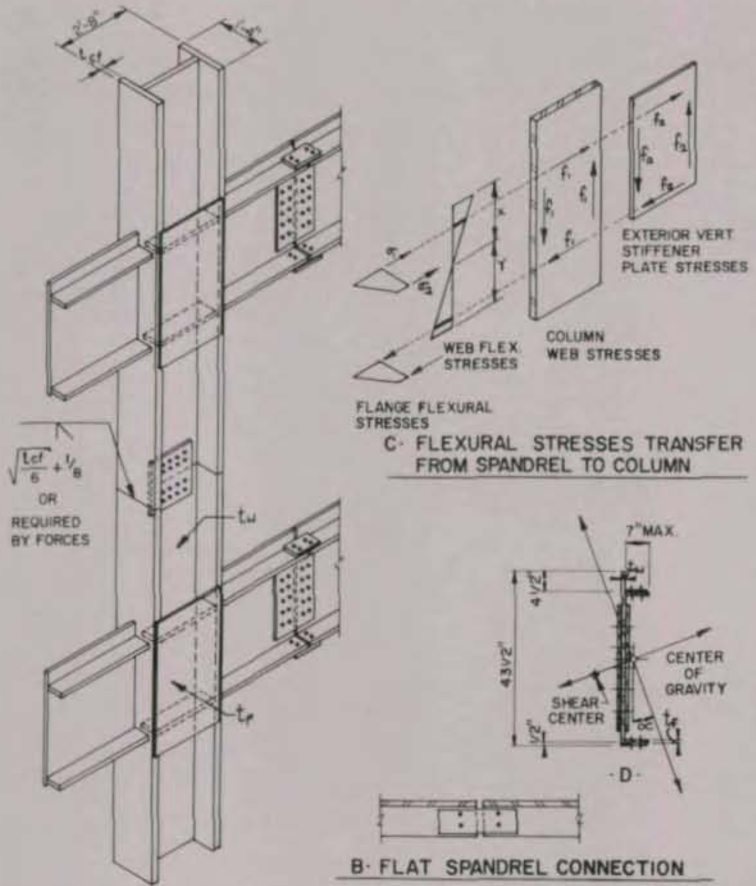


Figure 6 A· TREE COLUMN CONNECTIONS

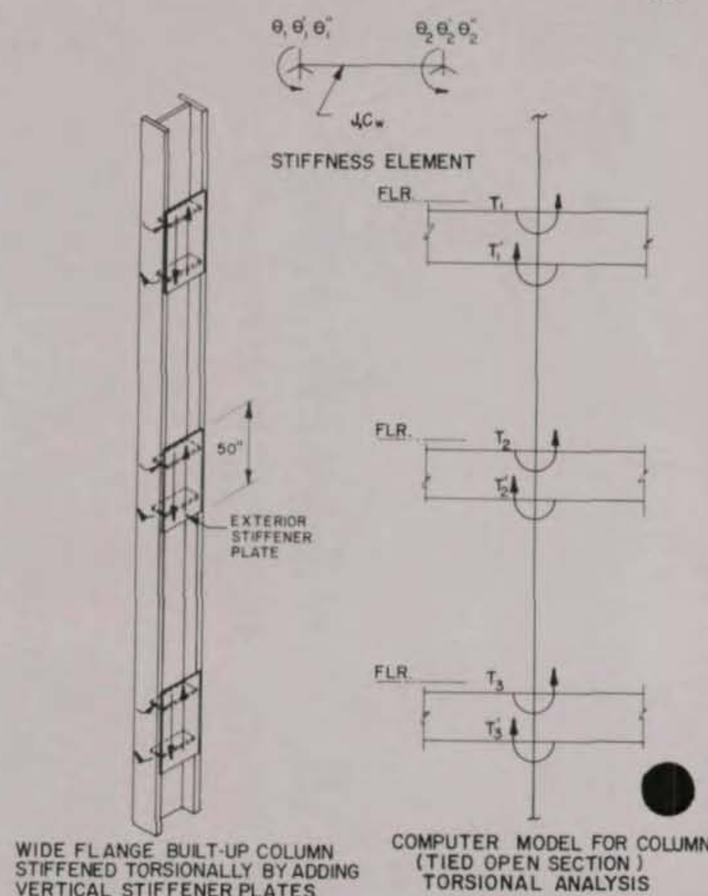


Figure 7

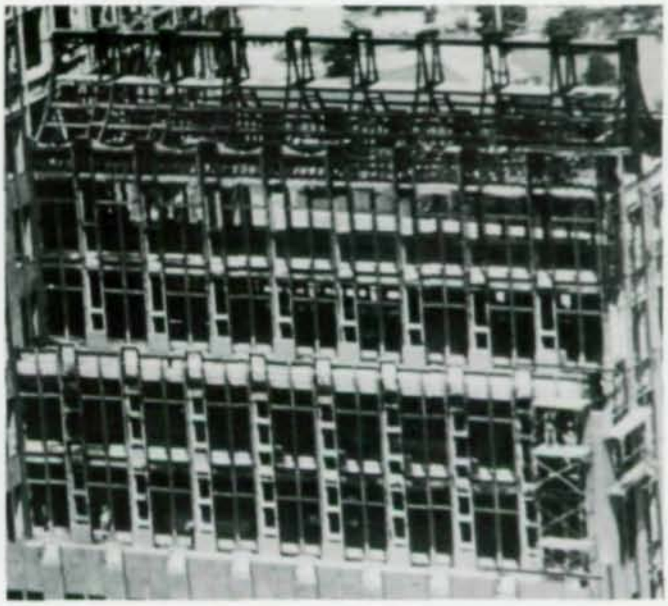
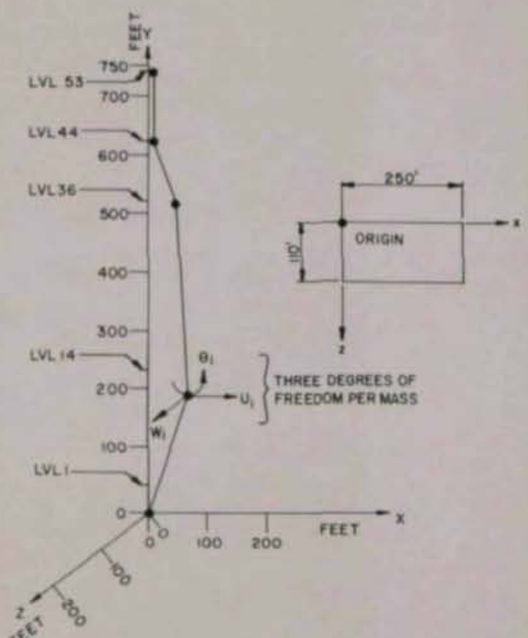
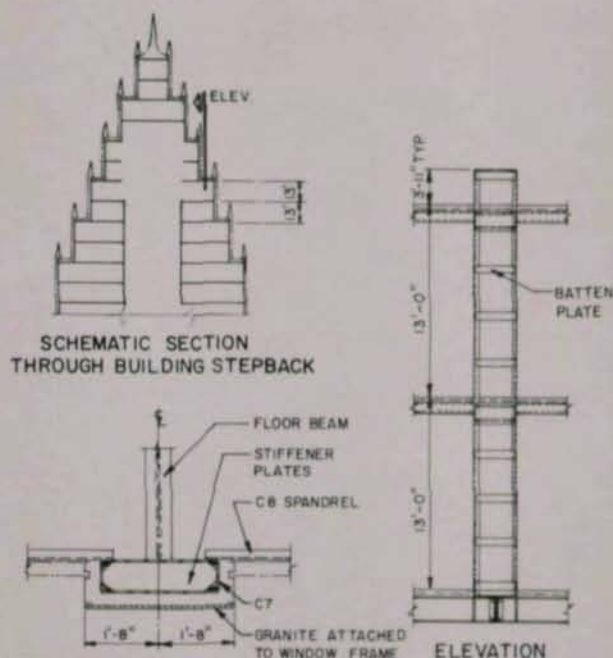


Figure 10



FOUR LUMPED MASS MODEL FOR DYNAMIC ANALYSIS

Figure 8



BATTENED (BUILT-UP) COLUMNS

Figure 9

joists spaced at 2.5 ft o.c. and spanning 110 ft. Its support system is a perimeter system with three of the four sides having spans of over 110 ft. The long spans were necessary because additional columns could not be introduced adjacent to the existing Western Union Building, except on one side and one corner only. All columns for this quadrant rest on deep drilled piers.

To match the lateral deflections of the two quadrants, a link arch was introduced at the east face of the structure (Fig. 12b). The north face (hybrid truss Fig. 12d) had to be tied laterally and supported off the main tower corner column. The west side was also tied horizontally at two levels to the main tower floors by in-plane diagonal bracing (Fig. 12a). A special stability analysis for the hybrid truss spanning 110 ft was conducted.

The building topped out in February of 1983 and initial occupancy began in the month of October 1983. □

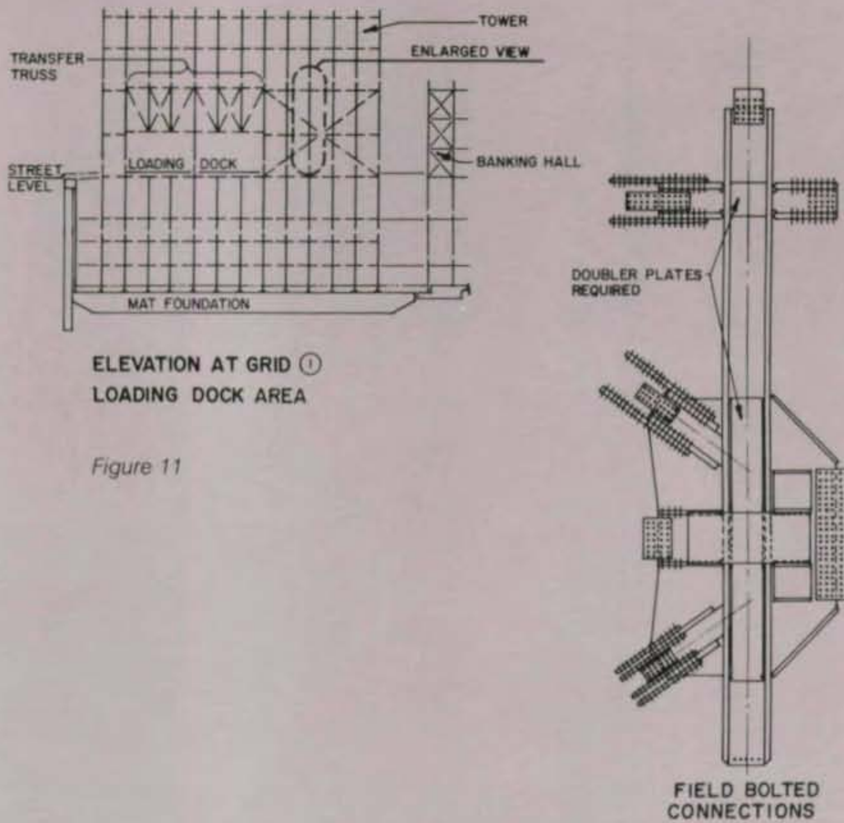


Figure 11

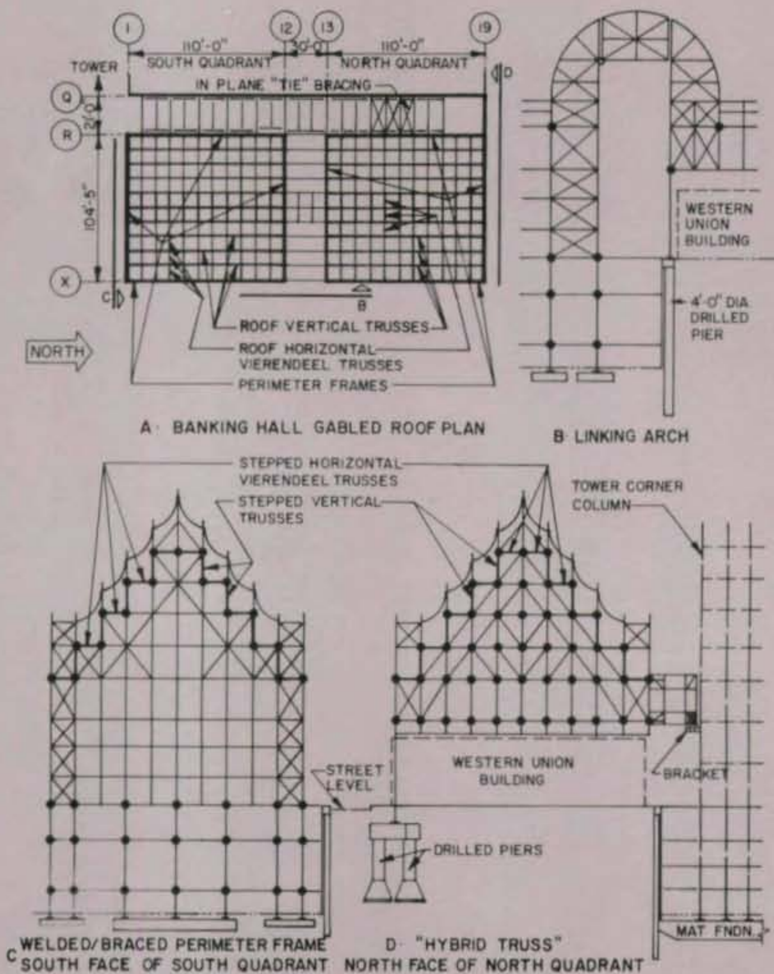


Figure 12

Architects
Philip Johnson/John Burgee Architects of New York, New York; and
Kendall/Heaton Architects, Houston, Texas

Structural Engineer
CBM Engineers, Inc.
Houston, Texas

General Contractor
Turner Construction Company
Houston, Texas

Steel Fabricator
Mosher Steel Company
Houston, Texas

Steel Erector
American Bridge Div., U.S. Steel Corp.

Owner
Gerald D. Hines Interest
Houston, Texas

General Electric's R & D: Steel Bridges the Gap to Tomorrow

**by James E. Coffey, Jr.
and
Noel Fagerlund**

Spanning a broad ravine with steel provided a solution to a problem at General Electric's corporate Research and Development Center project. This design was a response to unique site characteristics and diverse program requirements. The new facility is designed to accommodate GE's growing R and D functions at Niskayuna, N.Y. through 1990 and beyond.

Located on a heavily wooded bluff overlooking the Mohawk River, the existing 1.1-million sq ft complex occupied most of the geographical peninsula on which it is situated. A 500-ft wide, 60-ft deep ravine, a physical barrier to previous development, separated this peninsula, on the east side of the 580-acre site, from the undeveloped west side. To accommodate future expansion and also to preserve the natural beauty of the Center's environment, the decision was made to bridge the ravine. Now, what was an obstacle to growth provided new opportunities for linking old to new, and at the same time permitted GE additional growth potential beyond 1990.

Noel Fagerlund is executive vice president, and *James E. Coffey, Jr.* a structural engineer in the architect/engineering/planning firm of Smith, Hinchman & Grylls Associates, Detroit, Michigan.

The Design Objectives

General Electric set out several desires and needs for the architects and engineers to achieve. First, the new facility had to functionally reinforce, and integrate with, existing facilities constructed in 1947. This included the primary entrance, pedestrian circulation, building services and common use support services. In addition to functionally merging the facilities, the new project had to reinforce the architectural presence of the existing building complex—and also reflect the strength and vitality of GE's new research commitments.

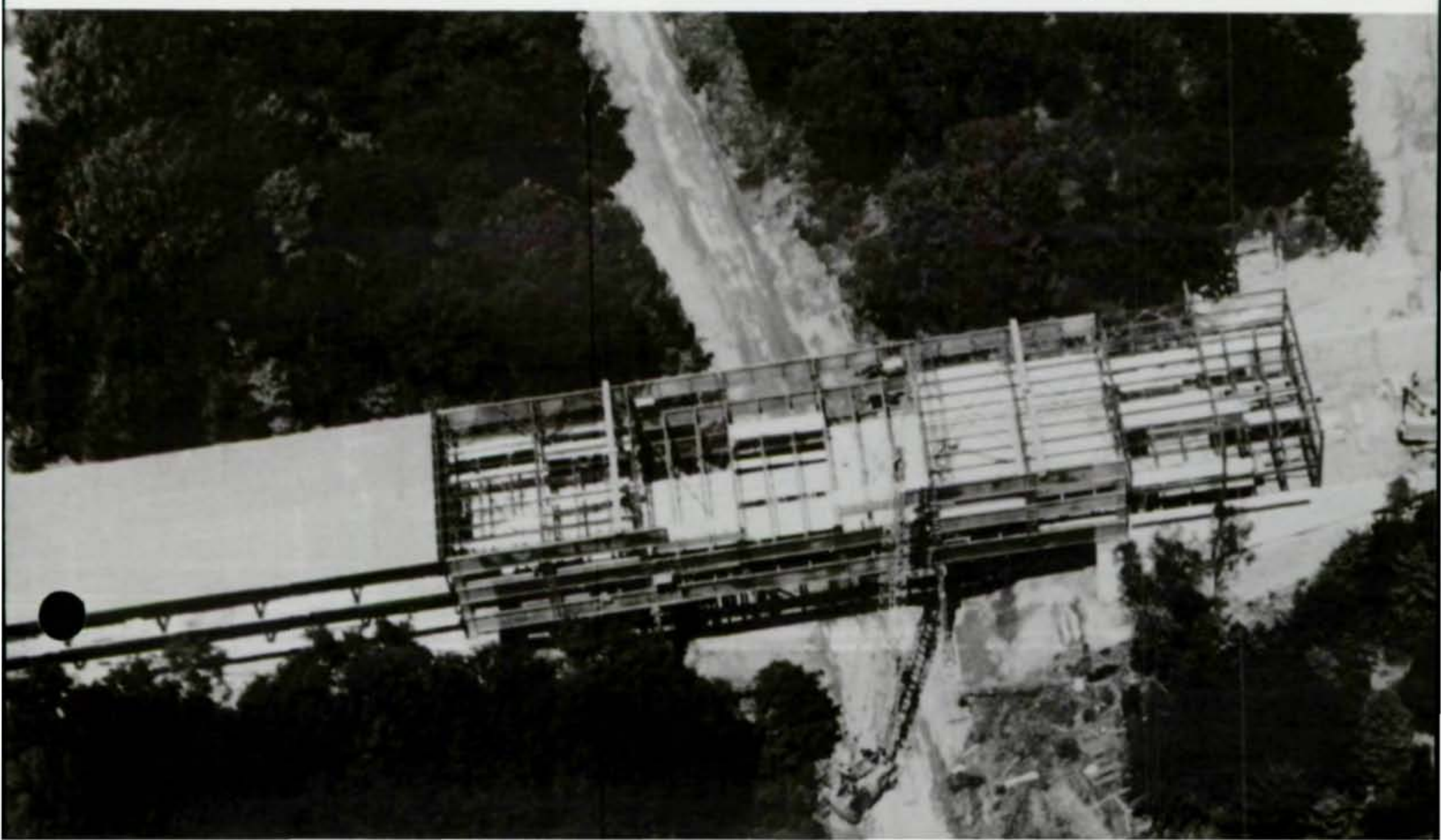
Second, the new facility had to be designed to allow the scientific staff to enjoy the environmental quality of the site. Also, it should encourage "informal" interactions among the technical staff, an essential communication ingredient in the scientific community. An attractive, functional facility is also considered essential to attracting and retaining talented scientists.

Third, the facility needed the flexibility to accommodate future incremental laboratory expansion, as well as respond to changes within the individual laboratory modules.

The Design Concept

The new facility encompasses three distinct, but contiguous, elements. The *Commons*, which serves both new and existing facilities, houses the library, cafeteria, computer center and graphics operations. The *Commons*, along with the atrium and

General Electric's new R & D Center, Niskayuna, N.Y. Steel bridges the gap to tomorrow for growing facility.



the existing building's main entry/lobby and new conference center, form the epicenter of the entire R and D facility in terms of function, order and image.

The *Laboratory*, based on a new generation of laboratory modules for electronics research, required a new and separate identity, as well as physical adjacency to the existing complex. This element, with its inherent expansion requirements, belonged across the ravine where there was ample land and the opportunity to present a new identity.

The *Office* contained functions that related primarily to the new laboratory element, and secondarily to existing management functions. The *Office* element was placed between the *Commons* and *Laboratory*, providing the required functional relationships as well as bridging the ravine, thus satisfying the need to link the complex into one unit.

The Commons

A three-level atrium connects the new Commons to the existing building. It serves as the heart of the facility and forms a link between the new and the old. A spectrum of common use functions radiates from the atrium, reinforcing it as the meeting place for the Center scientists and the international scientific community attracted here. Built on a hillside, the five-story Commons building is faced on three sides with a buff brick to match the existing structure. The fourth side, overlooking the ravine and the new laboratory and office complex across the ravine, is faced with white precast panels with limestone aggregate and reflective glass ribbon windows.

The top level, Level F, of the Commons building holds the library, stocked with scientific publications from all over the world. The library floor is designed for 150 psf live load. The next level down, Level E, contains the cafeteria and kitchen areas, designed for 100 and 165 psf, respectively. This level meets the grade level floor of the existing building. Level D, the bottom level of the atrium, coincides with the top level of the ravine-spanning office, and houses the graphics department. Level C, the computer level, is slab-on-grade construction. The main corridor for traffic to and from the new laboratory building enters the Commons building at this level. Level B is a mechanical basement that extends into the ravine under the Office building.

The interface between the existing building and the new building presented some complex structural problems. The existing structure, built to the edge of a hill, used spread footings which did not accommodate future expansion. A massive concrete grade beam/spread footing system, constructed in the hillside, culminated in a series of 5-ft deep, 11-ft long cantilever grade beams that reach to the existing building wall. An 85-ft long, 16-ft deep truss supports the roof and Level F floor at this junction with the existing building wall, allowing for open spaces around the atrium and easing the foundation problem. The ends of the truss rest on columns supported by the grade beam system.

The Laboratories

The new Laboratory building is organized around a major pedestrian and utility spine at the western edge of the ravine. The curving configuration of the spine avoids the "endless vista" along its initial 400-ft length. As additional lab units are built at either end, this optical reduction will become even more important. Laboratory modules literally plug into this spine in terms of utility distribution and secondary pedestrian access. Expansion will take place at either end of the Laboratory building by simply extending the spine and plugging in more modules.

Since research, by its nature, is unpredictable, all of the laboratory modules have maximum flexibility for change and adaptation. Spaces can be made larger or smaller at will, taking advantage of the large steel-framed 30-ft by 45-ft bays. Utility systems were also designed for ease of change. Corridors and offices flank either side of the wide laboratory spaces.

The pedestrian corridor spine that curves along the ravine emphasizes the wooded natural environment of the ravine. Two-story high glass exterior walls open up this space to people on both floors of the Laboratory building. Trusses span 45 ft between columns to permit a relatively unobstructed view. These trusses also support the beams that make up the floor of the mechanical penthouse spine directly above the corridor. An extensively landscaped courtyard in the center of the Laboratory building serves as an entrance and also provides aesthetic relief to the strong functional nature of the labs. The reflective glass and white precast panels face the ravine, mirroring the architectural treatment of the Commons building. For the entrance side of the Laboratory building, brick masonry, the site standard, was used. Metal siding was employed on the north and south walls where future expansion will occur.

The Office

This element was designed as a link between the existing complex on one side of the ravine and the new complex on the other. The reflective glass tube mirrors the natural environment of the ravine. Two levels of offices sit atop a mechanical space that carries all utility services from the main plant to the new Laboratory building. As well as the major utility connection, the Office building houses the major pedestrian connection between the old and the new. This circulation link is located along the south face of the C level, offering panoramic vistas of the natural terrain. Offices at D level are open-plan, with maximum flexibility of space utilization provided by the 55-ft wide, column-free floor space and 3-in. electrified floor system on a 5-ft grid.

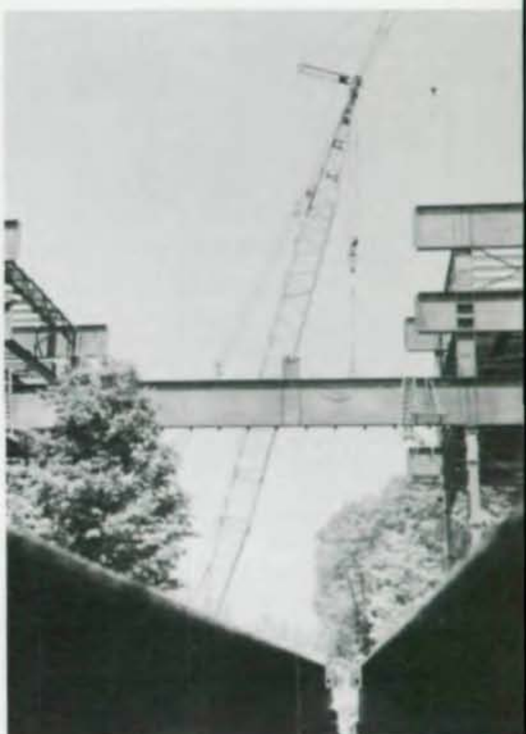
Of the 500-ft total length of the Office, 210 ft clear-span the ravine with a two-story Vierendeel truss framework. The remainder, 160 ft on one side and 130 ft on the other, sits atop reinforced concrete walls along the sloping sides of the ravine. Mechanical and electrical equipment is housed in each of these semi-basements, which in turn feed into the plenum and pipe space in the underbelly of the office space. Throughout the 500-ft length of the Office building, 55-ft long composite floor beams spaced 10 ft o.c. span the entire width of the building. At the roof level, 6-ft deep trusses span the building. At the soffit level, a 6-ft 6-in. deep mechanical plenum, WT sections are hung from the floor beams above, and 3-in. metal roof decking spans them.

Challenging Construction

The most challenging aspect of the Office building construction was the fabrication and erection of the 210-ft long, 39-ft tall Vierendeel trusses. Truss verticals were 36-in. and 42-in. H-sections built up from plates. Splices made at the mid-point of each vertical truss member made each element similar in appearance to a "tree-beam." The three-level truss used 8-ft deep plate girders at the top and bottom and a 7-ft deep girder in the middle. Splices placed in these members permitted the truss to be fabricated in three 70-ft long sections. One Vierendeel truss was employed on each side of the Office building, with the 55-ft floor members and roof trusses framed into the plate girders. Ends of the two trusses rest on pot bearings, which in turn sit on a W14x233 section embedded in the reinforced concrete abutments.

To erect the truss elements, temporary shoring was placed near the splice locations to support the end truss sections. The bottom chords of both sides of one end of the clear span were erected first. Floor beams for the first level were then placed between these truss bottom chords. Next,

Aerial of finished structures. Bridge (l., top l., above & r.) accommodated expansion to west and preserved beauty of environment, providing future growth opportunities beyond 1990.



the middle chords were placed atop bottom chords, vertical splices were made and the floor beams placed. Finally, the top chords were placed and welded, then connected by roof trusses. The same erection procedure was used on the opposite side of the ravine. In erecting the center section of the span, close measurements were taken and adjustments made to positioning of both end sections. When the center section was set, it fit like a glove. The Vierendeel trusses were fabricated and erected to allow for 2 in. of dead-load deflection and an additional 1 in. of live-load deflection. Each truss weighed 160 tons, or 1,500 lbs./ft.

Flexibility for the Future

A total of 2,100 tons of steel was used for this 340,000 sq ft Phase I of General Electric's Master Plan. The plan calls for an additional 160,000 sq ft of laboratory space through 1990. Since Phase I was completed, one laboratory module has already been added.

By using steel to bridge the gap, and steel framing in their Laboratory building, General Electric prepared itself for the future. The research of tomorrow can now be done in the buildings of today! ☐

Workers fasten huge steel beams that link complex into one unit.

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

General Contractor
Turner Construction Company
Philadelphia, Pennsylvania

Fabricators/Erectors
General Steel Fabricators, Inc. (Laboratory and Commons buildings)
Latham, New York

American Bridge Division, U.S. Steel Corp. (Office building)
Elmira Heights, New York

Owner
General Electric Company
Schenectady, New York

From Coast to Coast, Condos are Going Steel. . . .

IN VERMONT. . .

Mountain Green: Condominiums by Christmas— with Steel

by David T. Biggs and Stephen J. Sopko

Killington Mountain in central Vermont is one of the major ski areas in the eastern U.S. Because it has a unique combination of natural snow, extensive snow making and a northern exposure, the Killington ski season now runs from mid-October to mid-June, the longest in the East. In addition to its being a winter resort area, Killington has exciting activities in the summer and autumn seasons which atrract many tourists to the area year 'round. As a result, the condominium market has been growing steadily.

One of the most recent developments in Killington is the Mountain Green Golf and Ski Resort Condominiums. The complex of three buildings is located at the base of Killington Mt. within walking distance of the ski lifts. The multi-use complex provides residential units, retail shops, swimming pool, racquetball courts, restaurants and a 60-car garage. The buildings are oriented to provide a spectacular view of the area.

Nearly all condominium construction in Vermont is two-story, wood-framed buildings. For the few four-story buildings, masonry bearing wall buildings are usually constructed. Mountain Green is one of the few to use multi-level construction and structural steel. The design and construction of the resort proceeds in four phases. Each has been fast-tracked—foundations start in the spring, steel erection in the summer, and units are occupied in December.

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Special Conditions

Local building and fire codes limited the height of the buildings so that every floor is a maximum of 35 ft above finished grade along one side. Building 3, for example, has a 20-ft differential in exterior grade between the front and back. This difference in grade changes along the ends of the building. To accommodate the code requirements and maximize space usage, the buildings have basements below grade to contain mechanical rooms, a parking garage and racquetball courts in Building 3. The upper levels were stepped, and loft units incorporated into the design.

Buildings 1 and 2

A seven-story steel frame supporting precast plank floors (Figure 1) was selected as the structural framing system for Buildings 1 and 2, for a variety of reasons. In addition to being the most economical system, the steel framing system permitted the project to be fast-tracked, yet provided flexibility in design. Foundation construction and steel fabrication were completed before the architectural design was complete. By using steel, upper level lofts could be developed architecturally after the framing was installed. In Building 2, an additional floor was added, which included loft units.

Architectural floor plans were not suited to regular bays. But steel framing created the flexibility to incorporate numerous corners and column offsets throughout the structure. With the plank-steel system, an 8-ft-10-in. floor-to-floor height was used. Where possible, steel beams were aligned with partitions or placed in low ceilings in kitchen and bathroom areas, thus allowing an 8-ft clear floor-to-ceiling height in living areas.

An alpine sloped roof of 30° was incorporated into the design to keep the building in character with the area. The steeply sloped roofs with numerous penetrations for skylights and chimneys were suited to steel framing. A combination of structural steel and light-gauge steel joists were used to frame the roof. For economy, the developer selected prefabricated fireplaces for each living unit. To speed construction, the chimney frames were shop-fabricated from structural steel angles and WF and erected along with the building framing (Figure 2).

The most important considerations in choosing the structural steel system were cost and the speed of construction. Since the Vermont building season is relatively short, erection time had to be reduced to allow units to be occupied by the December holiday season.

Buildings 1 and 2 are essentially the same in design. Additional loft units were built in Building 2 with a composite floor deck, cast-in-place concrete and composite beams. The structure was designed using A36 steel. However, some higher strength steel was substituted to improve delivery time. The structures use conventional braced frames to resist the lateral loads.

Building 3

Building 3, a larger, more complex building with eight levels, has its own unique design (Figure 3). Buildings 1 and 2 had somewhat regular floor plans for different levels, making the use of concrete plank economical. In Building 3, each floor is different. Upper floors are incorporated into the 45° alpine-sloped roof area, which made it necessary to use stub columns and transfer girders to support upper floors and part of roof structure (Figure 4).

To reduce the dead load of the structure and have the flexibility to frame the upper floors, a steel joist system supporting a

2½-in. slab on galvanized forming was chosen. Floor-to-floor height was increased to 9 ft-3 in., but a 15-in. structural depth was required, except for main girders which had to be soffited. Since joists were required to span 24 ft, and the maximum joist depth was limited to 12 in., the height-to-depth ratio was in the critical range for floor vibrations. A vibration analysis of the floor system was performed to determine dynamic characteristics. It indicated the total mass of the finished construction, including walls and ceiling, would dampen the system, and vibration would not be a problem except in a few open commercial areas. In these areas, steel beams and a 5-in. slab were substituted for the joist system.

A 6½-in. slab on metal deck and beams were used at the first structural level. This system was chosen for two reasons. First, it provided a 2-hr. fire separation between the parking level and the main building. But more importantly, it provided a diaphragm to transfer earth and rock pressures from the high backfilled wall in front of the building to perpendicular transverse foundation shear walls, thus permitting a more economical foundation design.

Based on the experience of Buildings 1 and 2, the perimeter columns in Building 3 were separated from the building foundation walls.This greatly simplified construction details on the sloped site for both the foundation and the steel frame. Varying height piers and corbels to support steel were eliminated. This speeded the foundation wall construction time. The separation of walls and frame also allowed the foundation wall construction to proceed independently of the steel erection.

Subsurface conditions of each building were different. Building 1 is on spread footings on undisturbed soil; Building 2 is partially founded on the same undisturbed soil and partially on ledge. Foundations for Building 3 bear fully on ledge, which had to be blasted and excavated for basement areas. A system of spread footings, basement wall and grade beams supports the vertical and horizontal loads.

Summary

Due to architectural layout, it was not possible to brace the building conventionally. Type-2 framing with wind connections was used throughout. Each building was steel-framed with light-gauge metal stud exterior walls and STO energy system.

The use of steel as the framing system in the Mountain Green Condominiums resulted in an economical system based upon flexibility of floor plan and speed of erection. □

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Structural Engineer
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Rutland, Vermont and Troy, New York

General Contractor
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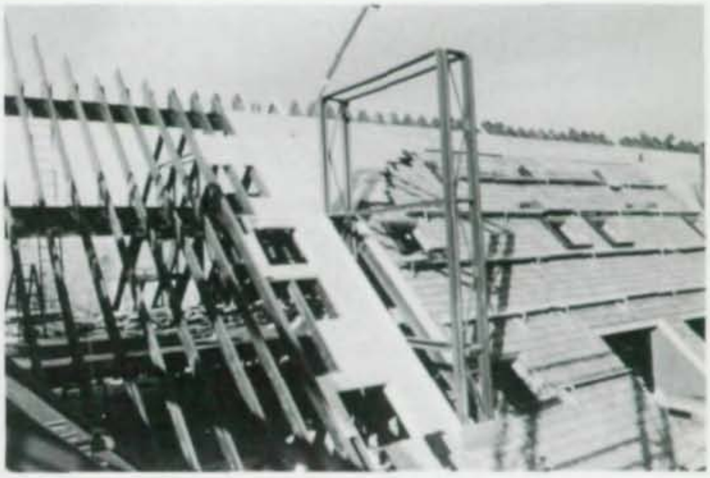
Mendon, Vermont

Steel Fabricator
Bennington Iron Works, Inc.
Bennington, Vermont

Developer
Mountain Green Associates, Ltd.
Killington, Vermont



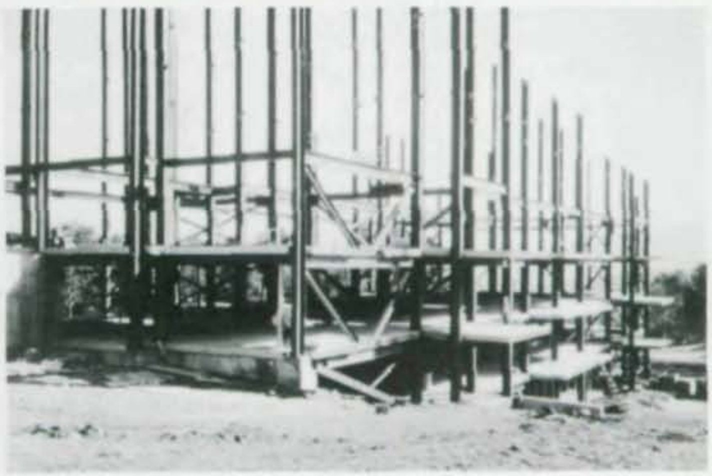


Fig. 1. Steel framing (top) permitted fast- tracking, yet provided for flexibility in design.

Fig. 3. Larger Building No. 3 has own unique design (bott. photo).

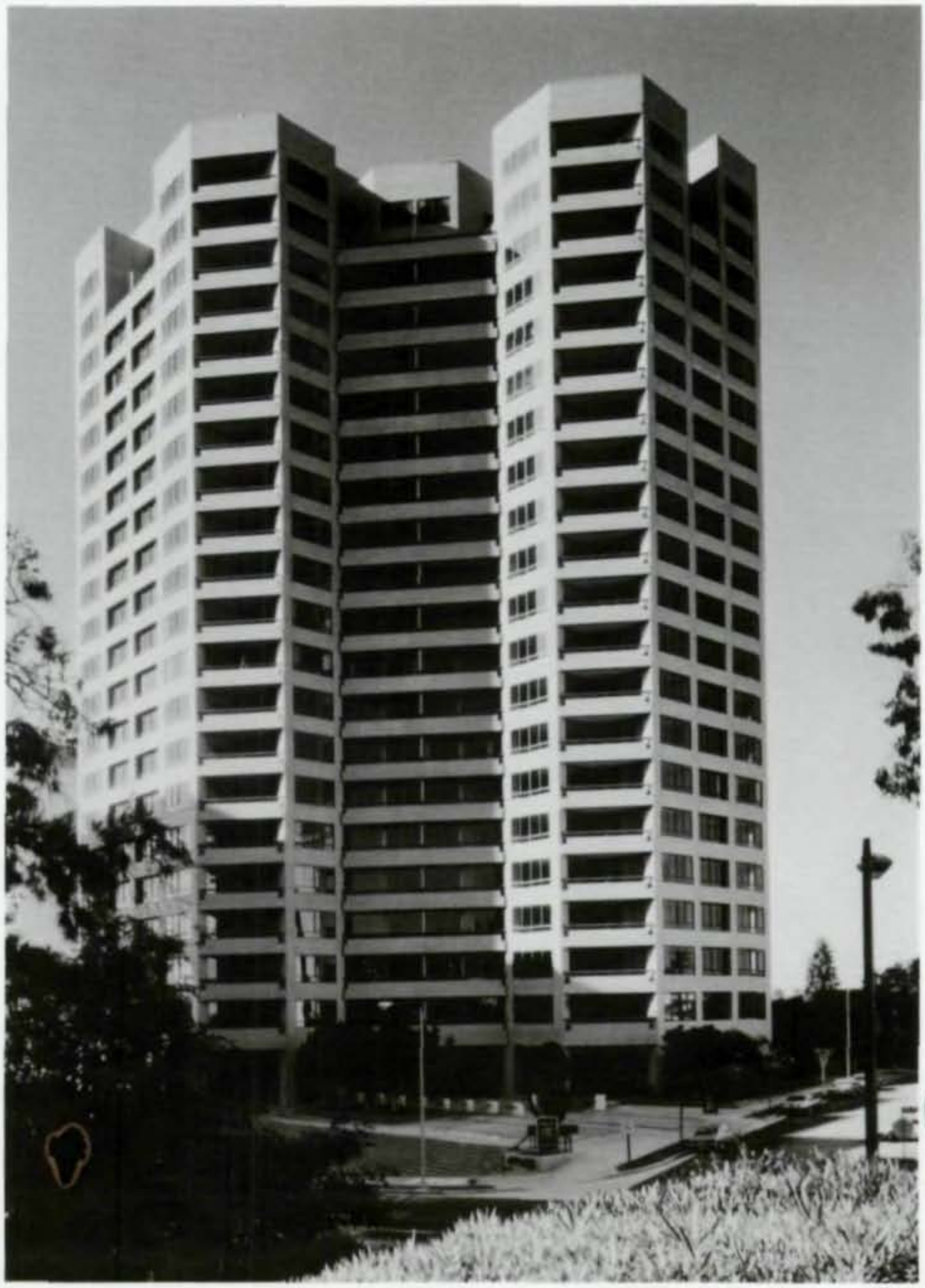
Fig. 2. Chimney frames were shop- fabricated and erected with framing to speed job (top).

Fig. 4. Stub columns and transfer girders were used to support upper floors and roof (bott. photo).

The Mirabella Condominiums: Innovation in Structural Steel/Concrete Design

by Arthur Yohannan and Barry S. Schindler

The Mirabella is a 21-story steel-framed condominium along Wilshire Boulevard's famous Golden Mile. Its distinct design, an innovation in traditional multi-family dwellings, creates a unique angular appearance. Although the building's exterior appears to be masonry, the structure is actually built of structural steel and concrete, with only a masonry fascia.



Design and Cost Efficiency

Influencing the design and type of construction were several factors and requirements which lent themselves to the use of steel: namely, code requirements for a ductile-frame design, cost considerations and solutions to high-rise structures in high-risk seismic zones.

Code requirements demand that buildings over 160 ft high allow for a ductile-frame design, which is relatively easy to achieve by using steel instead of concrete. In concrete, the size of core members, especially columns, becomes overbearing to the extent that floor space within the building is compromised. Steel was also a determinant in cost efficiency, since steel tonnage and fabrication were less expensive than concrete at bidding time.

The construction type of one wythe of brick is outside of a waterproof membrane fastened to structural steel studs. The entire assembly is attached to the structural frame.

Arthur Yohannan is a vice president of the architecture, planning and urban design firm of Maxwell Starkman Associates, Beverly Hills, California.

Barry Schindler is project manager of the structural engineering firm of John A. Martin & Associates, Los Angeles, California.

Seismic Consideration

Location of this high-rise structure is in Seismic Zone 4, which requires both the lightest building possible and reductions of the lateral systems. The lateral force-resisting system for the tower structure involves the use of ductile moment-resisting steel space frames. The economy of the system is enhanced by the nature of the frames in plan, which form two interlocking steel tube structures.

The state-of-the-art in lateral force-resisting systems are structural tube forms that provide the most economical solution for high-rise buildings in high-risk seismic zones. Lateral forces are transferred through the steel tower frames to the first floor, where the forces are then transferred into reinforced concrete shear walls via a reinforced concrete and metal deck floor system. These forces then are carried down the shear walls through the parking garage, and unloaded into the soil by passive pressure against the soil, and soil-concrete friction. Structural steel frames are designed to withstand the maximum probable earthquake, while the reinforced concrete shear walls are designed to resist the more severe maximum credible earthquake, since concrete is not as ductile as is structural steel.

Structural System

Vertical loads in both the condominium and parking garage are supported by filled metal deck spanning structural steel

Mirabella Condominiums (l.) on posh Wilshire Boulevard's Golden Mile in Los Angeles. Distinct design, cost efficiency and seismic requirements dictated steel frame (below).

beams. Both metal decking and steel beams incorporate the design feature of composite construction in which concrete acts integrally with steel. Steel beams transfer vertical loads to steel girders and columns. The columns are designed based on a higher grade of steel than the beams to increase the efficiency of the structure. All vertical loads are transferred into the ground by reinforced concrete spread footings under columns and continuous footings under walls. Perimeter wall footings are designed as boundary conditions to facilitate construction by not interfering with soldier piles used for shoring the basement excavation.

Because the project was fast-tracked, it was imperative the structural frame of the building be erected as quickly as possible. This resulted not only in time savings, but also in the ability to keep up with the schedules of consultants and the subtrades. Problems and changes inherent in fast-tracking were mitigated and resolved by using steel, because of its flexibility and functionality over other materials.

Features

The Mirabella's exterior is a light salmon-colored brick veneer and bronze-tinted glass. Pedestrian access is via a monumental stairway from Wilshire Boulevard. A spacious European style brick porte cochere' motor entrance provides ingress to the lobby, which is accentuated with a black granite floor, greenery, brass and walls of glass. All 108 units have two bedrooms and two and one-half bathrooms, and range in size from 2,076 to 2,862 sq ft. There are also six two-story penthouses with two to four bedrooms. Every unit has been uniquely designed in a corner position to maximize multi-directional views of the surrounding mountains, coastline or cityscape. Spacious terraces and full-height windows and doors enhance the scale of panoramic vistas. □

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General Contractor
Swinerton & Walberg Company
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Owner
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Steel-framed structure on fast track for time savings and flexibility for making changes.

Handsome exterior of brick veneer and bronze-tinted glass adds angular touch to luxury building.

Steel-Framed Parking Structures in Up Trend. . .

Here are 10 Tips to Maximize Efficiency, Service Life

When the need arises for a multi-story parking building—by municipalities, airports, hospitals, downtown stores, colleges, etc.—parking authorities find increasing interest in the steel-framed, open-deck parking structure.

John Fujiwara, president of Des Man Parking Associates, New York City, which specializes in design of parking structures, says: "While we're impartial as to choice of construction materials, steel certainly has its place where conditions warrant it, and can often bring substantial economies to a project. We designed three garages in Portland, Me., and the speed of erection of steel framing made it the choice because of the area's short construction season.

"We also used structural steel in a 530-car garage for the St. Francis Medical Center, Trenton, N.J. Here, neighboring homeowners demanded an attractive structure that screened parked cars from view, yet did not appear out of scale with the surrounding residences. Our design resulted in a handsome, five-level garage with a bronze-tone metal grille screen that has pleased everyone."

A strong trend to the open, steel-framed structures is reported by Joseph F. Mulach, Jr., chairman of Mulach Parking Systems, Bridgeville, Pa., a leading design/build firm specializing in parking garages. Mulach states that a new development is their placement in dual-occupancy buildings, featuring a garage either above or below commercial or residential space. The firm believes a steel frame with post-tensioned concrete slab floors offers the maximum durability which owners and developers seek in multi-level parking buildings.

Says Seymour Gage, of Seymour Gage Associates, architects/engineers in White Plains, N.Y.: "We have already designed 25 steel parking structures, including the tallest ever built in steel, an 11-story, 1,386-car unit here in White Plains. We cut costs $800,000 on it by switching from precast concrete to steel. Now, we are also working in the retrofit market, taking old steel facilities and bringing them up to date functionally and aesthetically. Europe, incidentally, is way ahead of us in erecting steel-framed garages. National Car Parks, in England, has constructed 800 garages, with a substantial number in steel."

According to parking professionals, three significant reasons account for the trend:

- Generally, steel-framed parking facilities are now more economical to construct than competitive framing systems, although cost is always dependent upon varying market and local factors. This economy takes on added importance when considering that the frame, foundation and floor slabs normally constitute 60% of the costs of these buildings.
- Steel's potential as a prime structural material for parking purposes really commenced in the mid-1970's—when building code authorities permitted exposed steel construction, at first for small, low-rise garages, then for larger and taller facilities. The three major code organizations—Building Officials and Code Administrators, the Southern Building Code Congress and the International Conference of Building Officials—all approve exposed steel framing. Most regions, states and municipalities follow their guidelines.
- Thirty and 40 years ago, the image of steel parking structures was strictly utilitarian . . . i.e., buildings performed their function with little or no regard to aesthetics. Now, their appearance is second to none, the result of architect designs and the ability of steel framing to be combined attractively with a variety of exterior materials.

Essentially, the system is based on a clear-span, structural steel frame with girders spanning transversely 55 to 64 ft apart to provide maximum flexibility of layout. The steel most frequently employed is a high strength/low alloy, columbium-vanadium steel, ASTM A572, Gr. 50, with a

Located at popular Pittsburgh office, shopping, entertainment complex, 815-car Station Square garage uses exposed steel as prominent facade. Architect: Landmark Design.

Eleven-story steel-framed parking structure in White Plains, N.Y. serves adjoining 23-story office building. Switching to steel saved $800,000. Architect: Seymour Gage.

yield strength of 50,000 psi and a tensile strength of 65,000 psi. Another commonly used construction steel grade is A36. With 320 sq ft considered as the average parking space size, approximately 1 to 1¼ tons of framing steel are used to support each space, 7½ psf.

The key durability factor in a multi-story, open-deck parking structure is the floor deck system. Proper slab design is the best assurance of an extended service life; otherwise, the floor slab can literally be the weakest element in the overall structure. Three popular options are now available.

One is the cast-in-place, concrete *post-tensioned slab*, 5-in. to 6-in. thick, which encompasses tendons of steel wire sheathed in plastic, and post-tensioned in both directions to offer the closest to a crack-free slab system. Post-tensioning compression tends to minimize or close shrinkage cracks in the concrete, thus limiting the entrance of chloride-laden moisture from deicing salts.

A second deck system is a cast-in-place, *reinforced concrete slab*, usually 4½-in. or 5-in. thick, with steel bars as the reinforcing agent. In snow/ice areas where deicing salts are heavily used, a minimum 5-in. thick slab is often chosen to provide extra protective coverage against corrosion of the rebars.

Up-and-coming in favor is the precast, prestressed *concrete wideslab* deck, 8-ft wide and 2¼-in. to 3-in. thick, with approximately 3 in. extra of supplemental concrete added in an on-site pour. Ultimately, both thicknesses act as a composite unit.

10 Tips to Greater Efficiency, Longer Service Life

Here are some proven practices to heighten the functional effectiveness and *durability of steel-framed parking structures*.

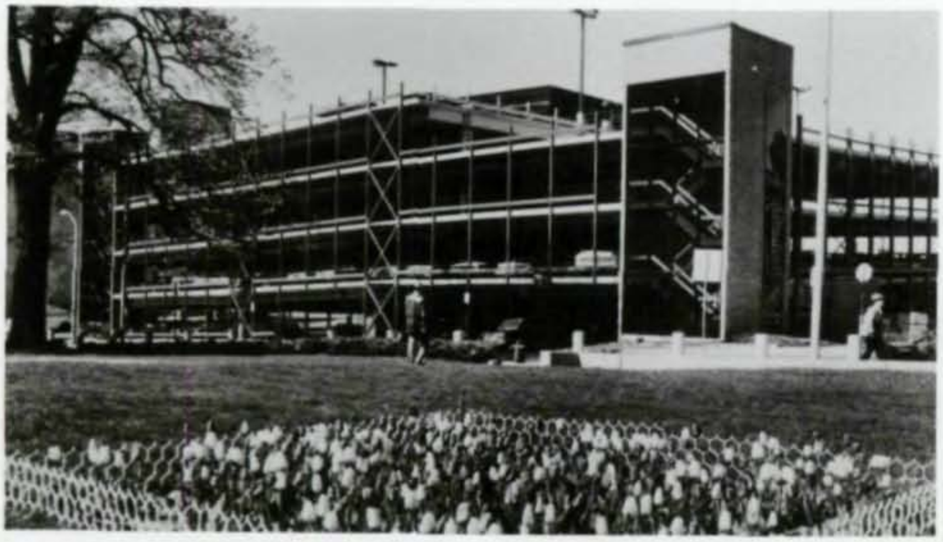
1. All concrete floor decks expand and contract with temperature changes, sometimes extreme over the course of a day. To minimize cracking, this movement must be accommodated. It is best done by pouring concrete into independent deck sections, separated from each other by expansion joints or slip planes. Any movement that does occur is absorbed by the thermally responsive joints and the flexibility of the steel frame. Volume changes in one slab will not affect adjacent areas.

2. Avoid having floor areas where water can collect into ponds. Deicing products will collect with the water and start the salt penetration process. To eliminate ponding, all deck surfaces should slope to drains—a ¼ in. in 12 in. minimum slope is desirable. Locate floor drains at the ends of the parking spaces to prevent surface water from draining over the framing members.

3. A related problem, common to any structure using improperly designed concrete floor slabs is spalling—delamination, potholes and other defects—traceable directly to corrosion of the reinforcing bars embedded in the concrete deck. The cause is penetration of salt from certain concrete aggregates and deicing compounds. The proven solution is to use epoxy-coated rebars, where the coating is fusion-bonded to prevent chlorides from reaching the steel surface. Depending upon the deck system, these will mean an additional 15 to 25 cents per sq ft premium (one percent or less of the entire project cost), cheap insurance for the added protection against concrete deck failure.

4. The University of Akron used a weathering grade for all 1,000 tons of structural *steel in its 1,270-car, five-level facility, built* to help solve its campus parking problem. Also included was a weathering steel guardrail system. Although the steel is premium priced, life-cycle economies can accrue through elimination or minimizing of painting.

5. Improved, longer-lasting paint systems are now available to help protect the



Two upper steel-framed levels of Binghamton, N.Y. parking deck were added 20 years after original building, to add 200 more spaces. Architect: Seymour Gage.

Parking deck for Roanoke Memorial Hospital (above, r.) sits on 30° slope. Cars enter at upper level, exit below.

University of Akron parking deck (r.) uses inner weathering steel structural shapes and guard rail to reduce maintenance.

structural steel against corrosion. Mulach Parking Systems claims a 15-year life expectancy for its paint process: a shot-blast cleaning of the steel, followed by application of 2.5 mils of a high-solids, zinc-rich epoxy in the fabricating shop, then 2 to 5 mils of a high-solids epoxy polymide at the job site.

6. Those needing to construct a parking structure on a steep hill can emulate the 663-car garage for the Roanoke Memorial Hospital, in Virginia. On its 30° slope, located between two streets, the seven-story structure permits traffic to enter on the upper street, descend the multiple parking levels on a continuous ramp and exit onto the lower street. An elevator transports visitors directly from the parking levels to a pedestrian underpass which provides a safe and weather-free route to and from the hospital lobby.

7. To increase customer security, Detroit's 1,523-car First-Bagley garage designed each of four stair towers as entire grade-to-roof glass walls. Patrons using the lighted stairs can be seen from the outside, a popular visibility feature believed to hinder the incidence of crime.

8. Need more parking spaces in an all-concrete garage? Do what the City of Ann Arbor, Mich. did—add three steel-framed levels to straddle the existing, four-level structure and provide 388 additional parking spaces. Even more spaces are possible on the same frame, which was erected within the lot bounds of the existing structure. The City of Binghamton, N.Y., did much the same in a 200-space addition atop its three-level Collier Street facility.

9. Furthering a garage's profitability is accomplished by setting aside the grade floor for store rentals, as does the 506-car Temple Street facility in downtown Portland, Me. It has available a 12,500-sq-ft commercial area. Fire protection, required only for the steel columns and beams surrounding this commercial level, is spray-on fireproofing which provides a two-hour rating for these members.

10. From now on, design and construction of any parking garage should anticipate use by an increasing percentage of smaller cars. Some parking specialists project an 8 to 15% increase in parking capacity will be needed for the compacts of the 1980's and 1990's. This increased capacity might be achieved by reducing the parking module size to fit shorter, narrower cars, by painting stripes at a different angle, and by improving traffic flow patterns to take advantage of the shorter turning radiuses of smaller cars. □

Ann Arbor, Mich. 3-level steel-framed addition straddles existing 492-car deck, provides 388 more spaces.

We are indebted to AISI's Steel Products News Bureau for this information.

Trapp Family Lodge: The Hills are Alive Again!

Editor's note: *The sounds of silence must have fallen over the Stowe mountainside that December 1980 day as Baroness Maria von Trapp and her family stared into the smoking ruins of the Trapp Family Lodge. The tragic fire had consumed not only their beautiful lodge but also the memorabilia of a lifetime. Millions who have thrilled to the warmth and drama and heart-rending scores of Rodgers and Hammerstein's "The Sound of Music," will remember it as the life story of the von Trapps as they and their eight children left everything behind to flee the Nazi scourge. (This editor has seen it eight times.) Ultimately, they settled here and purchased a 1,700-acre farm overlooking the magnificent ver-mont (green mountains) of northern Vermont. What better reminder of their Austrian heritage and countryside! And over the years, they have developed a complex of lodges, a restaurant, skiing schools, a music camp at one time, and now time-sharing guest houses.*

Much of the von Trapp's lifetime has been devoted to new beginnings, and triumph over adversities. And the new lodge is really a third beginning for the Trapp family. The first guest checked into the new, and expanded, lodge on Dec. 16, 1983—almost three years to the day of that disastrous fire. A modern 73-room hotel rose from the ashes of its 27-room predecessor. The new lodge and guest houses, representing an $11.8 million investment, required one of the largest private financing arrangements ever made in Vermont.

Truly, the hills are alive again—to the sounds of music, and laughter, and sleighbells and schussing and all the other enjoyments that are a vital part of this bit of Austrian yesteryear framed in the architecture and structural technology of tomorrow.

by James A. Groom

James A. Groom, AIA is project manager of the architectural firm of Robert Burley Associates, Waitsfield, Vermont.

The new Trapp Family Lodge in Stowe, Vt. is a replacement of the original lodge which was destroyed in December 1980. The new structure not only had to meet current fire-safety codes and other regulatory standards but also it had to retain the warmth, friendliness and Austrian heritage of its well-known predecessor.

The challenge of the architect team was to meet these unique design goals on a fast-track schedule—and to do it all on an unusually tight budget for a first-class resort hotel. That challenge must sound familiar to every professional engineer and architect in the country. Perhaps the reputation of the client—the von Trapps of "Sound of Music" fame—heightened the sensitivity to the problems faced. Perhaps it was the fact that the project had such a high potential for design excellence. Who can say? The fact of the matter is that the team all felt the full weight of the responsibility.

For the structural engineers, the fast-track schedule meant they had to work in

New Trapp Family Lodge, Stowe, Vt. opened recently to replace previous one destroyed by fire.

reverse of the normal procedure. Contract drawings had to be produced from the basement up—without the benefit of knowing the final design of the upper levels. The foundation is not a simple rectangular foundation. The complex geometry of a building, which both terraces up a hill and has several 45° angles in plan, made the task of fixing dimensions for the footings interesting, to say the least.

It was so interesting, in fact, that for a three-week stint Frank Zamecnik, one of the engineering firm's principals, literally moved himself and the entire design team to Vermont to keep pace with the construction crews. Even with that effort, the shop drawings from the steel fabricator were used for a time in lieu of official contract drawings to keep the flow of information a few days ahead of construction crews. Not until documents were sufficiently ahead of construction did they pack their steel manuals and calculators and go home.

In the classic hurry-up-and-wait tradition, uncompleted drawings were packed into drawers while construction stopped for nine months to wait for financing problems to be resolved—problems brought on by the 1981-1982 high interest rates and difficult financing for even a traditional resort-hotel project. Completing the drawing after the long delay meant re-assembling the team and adding some new faces. Winter was near, and the time schedule became a critical factor once again.

New Lodge a Design Challenge

The new building was designed to terrace up a hill which ran behind the site of the original lodge. This concept was the idea of Johannes von Trapp and the architectural team all liked it immediately. The plan gave every room an excellent view, and produced an energy-efficient envelope. And it helped reduce the scale and visual impact of the structure. It also produced significant engineering problems!

The wet soils of the hillside made the building act as a type of dam. The architect had to deal with the resultant horizontal forces. Various ideas, such as running cables back to deadmen buried in the hill, were dismissed as inefficient, or not cost-effective. The final plan determined that the party walls between rooms, and the composite floor slabs themselves, could carry the loads if they were build of properly reinforced concrete. Since the upper floors did not have the same load to carry as the lower ones, a point was determined at which a conventional steel frame made the most sense.

The roof, with all those dormers, posed yet another problem. The traditional material for this type of intricate roof framing would be wood. But the 1976 *National Building Code* does not permit any combustible structural elements. Light-gauge steel framing was the next best solution.

Now, here is a 4-story, 70,000-sq ft building designed with numerous 45° angles, terraced into a wet hillside with unstable soils, and with five different structural systems—all on a fast-track

Trapp Lodge, a bit of Austria transported to Green Mountains, takes on new life with structural steel as major element in meeting safety requirements and design challenges. Photo by Michael Clark.

schedule—if you don't mind. Those systems included a conventional concrete frame, a composite floor system, load-bearing block, conventional steel framing and load-bearing light-gauge steel. The conventional steel framing and the roof framing of lightweight steel presented a very intricate network—and steel was the best way to serve it. Again, emphasis on the fact steel can accommodate a combination of irregular shapes, lack of duplication and intricate details. The architect/structural engineer team had worked together on several other projects, and they respected each's expertise—and are still working together.

Looking back, there were scores of design, engineering and technical team members who worked together on a daily basis for the four years it took to design and build the lodge. They are the ones who deserve the credit for executing the bold concepts in this structure, credit the principals receive. When all is said and done, perhaps the ability to pick the right team, and to attract and keep good people is that for which the architect really deserves credit. □

Maria von Trapp and son, Johannes, keep watchful eye on progress of new 73-room hotel.

Architect
Robert Burley Associates
Waitsfield, Vermont

Structural Engineer
Speigel and Zamecnik
New Haven, Connecticut

General Contractor
Pizzagalli Construction Co.
Burlington, Vermont

Steel Fabricator
Isaacson Structural Steel, Inc.
Berlin, New Hampshire

Owner
Trapp Family Lodge
Stowe, Vermont

First Baptist, Orlando:
"Crossing Over" to a New Environment

Transplanting a large and thriving church from a crowded downtown area to a spacious site in the outskirts requires some real master planning. The architect involved in the new First Baptist Church, Orlando, Fla., had his work cut out for him in designing a new site on 156 acres of suburban land along I-4. The huge, and progressive, master plan calls for:

- A 164,000 sq ft Worship Center for 6,000
- An Education Center
- Fellowship/dining hall for 1,500
- A 400-seat chapel
- Recreation Center
- School for Grades K-12
- Senior citizens' living center
- Recreational vehicle vacation park
- Mission plaza for grouped chapels
- Missionary housing
- A 2,500-seat amphitheater
- Lakefront recreation area
- Parking areas
- 10 acres of retention ponds

The design of this mammoth $15-million complex had to be responsive to the environment—the planting and climate—so representative of Florida. The major worship center complex, centrally located on the southern part of the new wooded site, has primary access to the south. Three vehicular entry points serve an on-site loop road. The loop feeds into numerous parking areas that surround the church complex to provide minimum crossing of the pedestrian and the vehicular traffic. Landscaping and gentle berms direct visual emphasis away from automobile areas toward the buildings. Dramatic views of the major structures are visible from the loop through the trees.

The visual focal point of the complex is a large cross and lantern with its stained glass elevated above the 60-ft high trees. This major element, bounded visually by trees, identifies the church building's presence. In response to the church's relocation theme of "Crossing Over," two major entries on each side of the complex are accessible over walkways which appear to bridge reflecting pools.

Design on Hub Concept

Design of the complex was generated by the main emphasis of the sanctuary as a central hub, out from which radiates connection to the education building, the dining hall and the media building on a fan-shaped horizontal concourse. The concourse corresponds to the fan-shaped sanctuary to provide a strong sense of direction that culminates at the central atrium and large entry foyer at each end of the concourse. The large atrium and concourse, with its fountains, skylights and greenery, is a natural place to congregate and a transitional space between inside and outside areas. From the three-story atrium, members will be able to look out over the plaza, amphitheater and pond. This visual tie is a key element in the overall master plan.

Closeness a Criteria

The 6,000-seat sanctuary was designed to keep attendees as close as possible to the pulpit to create the feeling of unity. The fan-shape concept was essential to this idea. The balcony reaches down on each side to create a sloped-floor lecture hall effect. The lower floor forms the center section below the center balcony. For emphasis, the baptistry becomes an integral part of the choir loft, without visually separating choir members. The choir loft, which wraps around the baptistry, orchestra and pulpit areas, blends into the congregational seating on the sides of the sanctuary to permit full expansion from 300 to 600 or more.

The Ultimate Solution—in Steel

The clear open space the owners required forms an oblique hexagon 330 ft across and 210 ft from the pulpit to the rear balcony. Many considerations were involved in handling the tremendous volumetric changes, especially during construction. The construction period over two winters was such that it would affect the structure before its own interior environment would control structure movement. Because of the fast-track schedule, many ideas required very early planning, with close attention to cost-saving solutions.

The ultimate solution involved long-span steel trusses—one main 140-ton truss girder spanning 180 ft and supporting four 70-ton secondary girder trusses that span an average of 170 ft. The main girder rests on two 80-ft high, 4-ft square columns. Long-span steel joists completed the roof support framing.

Since the roof forms a quadhedron, the front section was framed with two 90-ft un-

Rendering of new First Baptist Church, Orlando, Florida.

balanced scissor trusses that intersect at a point of maximum depth near the third point of the span. Erection required a common vertical truss member at the point of intersection. This permitted one truss to be fully erected and the intersecting twin to be assembled in two parts. A diaphragm was developed by combining 20-ga. welded roof deck and horizontal trusses between the secondary girder trusses. Shears were removed to the structure below by using a diagonally braced steel frame.

These main and secondary trusses were designed of 50 ksi A588 steel, which resulted in a 30% savings over A36 steel. Where compression or tension is prevalent, such as in the truss' elements, 50-ksi steel proved most economical because bending and deflection became less critical factors. The structural engineer says, "In most situations, a 30 to 40% less tonnage will be realized by taking advantage of the higher strength material. Caution, however, must be taken to ensure good welding quality and a compatible metallurgy."

Test of Patience

Erection proved a test of knowledge, experience and patience. The main girder truss was friction-bolted using load-indicator washers, and lifted (cautiously) 90 ft into the air by two 150-ton cranes. The 17-ft deep truss was then lowered onto anchor bolts, with only a ⅜-in. tolerance. In four hours, the truss was lifted successfully onto the 9 sq ft, 4½-in. deep bearing plates at each end. Movement is accommodated by Teflon-coated plates on cord-reinforced Neoprene pads. Once the main girder truss spanned majestically over the pulpit, the remainder of the trusses were assembled smoothly into place.

Phase 1, currently nearing completion, *contains the Worship Center, one educa-*tion building, a chapel and mechanical buildings. Soon the burgeoning congregation will be "Crossing Over" into their new, and magnificent, environment. □

Main 180-ft long span in final position. Truss is 18 ft-6 in. top to bottom, with chords of W14x 426 A588 steel.

Main truss supports four secondary truss girders (av. span 174 ft). Two 80-ft long scissor trusses frame opposite side.

Architect
Hatfield Halcomb Architects
Dallas, Texas

Structural Engineer
Gee & Jenson
Orlando, Florida

General Contractor
H.J. High
Orlando, Florida

Steel Fabricator
Owen Steel Company of Florida
Jacksonville, Florida

Owner
First Baptist Church
Orlando, Florida

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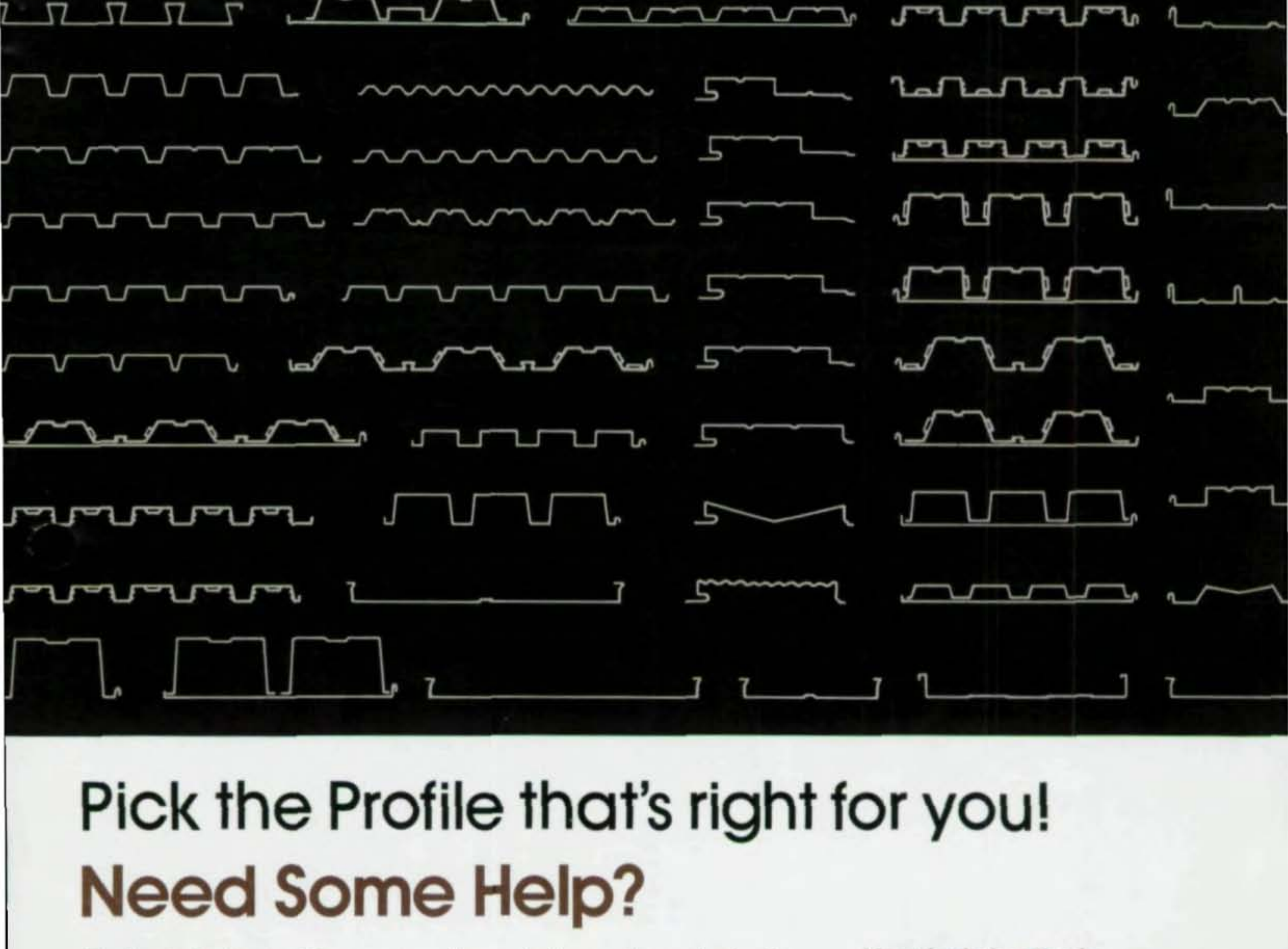

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