VOLUME XXVI NUMBER 3/THIRD QUARTER 1986

MODERN STEEL CONSTRUCTION

DECK DESIGN DATA SHEET NO. 8

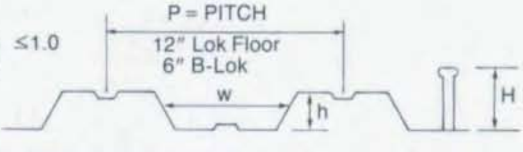
COMPOSITE BEAM DESIGN INFORMATION

1. Stud shear connectors should extend at least 1½" above the top of the deck.
2. The slab thickness above the steel deck should be at least 2".
3. Studs installed in metal deck can be placed as close to the web of the deck as needed for installation and to maintain the necessary spacing.
4. Deck anchorage may be provided by the stud welds.
5. For composite construction, studs should not be spaced greater than 32" on center.
6. "The minimum distance from the edge of a stud base to the edge of a flange shall be the diameter of the stud plus ⅛" but preferably not less than 1½"."—A.W.S.D1.1—79 Section 4.24.8
7. For many bay sizes it is advantageous to lay out the deck so that a deck valley lies over the center of the girder. For most composite girders there is no reduction in capacity if the deck is not split and spread apart at the girder.
8. N-Lok composite floor deck has a w/h ratio of 0.75 which makes it inefficient for composite beam design. N-Lok Cellular deck can be blended with 3" Lok-Floor and acceptable composite beam design can be obtained.
9. Designers are urged to check for the possible use of partial composite.

DESIGN VALUES FOR STUD SHEAR CONNECTORS (KIPS)

UNITED STEEL DECK, INC. SLAB TYPE			¾"⌀ STUD SIZE	w/h OF DECK	NUMBER OF STUDS PER RIB	RIB COEFFICIENT	ASTM C33 NORMAL (150 PCF) WEIGHT CONCRETE			ASTM C330 LIGHTWEIGHT (115 PCF) CONCRETE		
							3000 PSI	3500 PSI	4000 PSI	3000 PSI	3500 PSI	4000 PSI
SOLID CONC.			3"	—	—	1.0	11.5	12.5	13.3	9.9	10.8	11.4
MIN. SLAB DEPTH	3½"	B-LOK	3"	1.5	1	1.0	11.5	12.5	13.3	9.9	10.8	11.4
MIN. SLAB DEPTH	3½"	B-LOK	3"	1.5	2	0.9	10.4	11.3	12.0	8.9	9.7	10.3
MIN. SLAB DEPTH	4"	B-LOK	3½"	1.5	2	1.0	11.5	12.5	13.3	9.9	10.8	11.4
MIN. SLAB DEPTH	3½"	INVERTED B-LOK	3"	2.5	2	1.0	11.5	12.5	13.3	9.9	10.8	11.4
MIN. SLAB DEPTH	3½"	1½" LOK-FLR	3"	3.85	2	1.0	11.5	12.5	13.3	9.9	10.8	11.4
MIN. SLAB DEPTH	4"	2" LOK-FLR	3½"	3.0	2	1.0	11.5	12.5	13.3	9.9	10.8	11.4
MIN. SLAB DEPTH	5½"	3" LOK-FLR	5"	2.0	2	0.8	9.2	10.0	10.6	7.9	8.6	9.1
MIN. SLAB DEPTH	5½"	3" LOK-FLR	5"	2.0	1	1.0	11.5	12.5	13.3	9.9	10.8	11.4
MIN. SLAB DEPTH	6"	3" LOK-FLR	5½"	2.0	2	1.0	11.5	12.5	13.3	9.9	10.8	11.4

$$\text{Rib Coefficient} = \frac{0.85}{\sqrt{N}}\left(\frac{w}{h}\right)\left(\frac{H}{h} - 1.0\right) \le 1.0$$



N = Number of Studs Per Rib
H = Length of Stud
h = Height of Rib; 1½", 2", 3" Lok Floor
1½" B-Lok
w = Average width of Rib

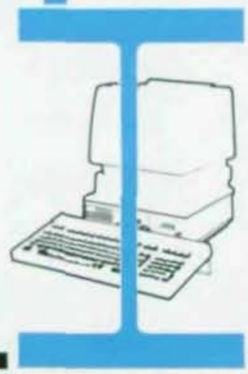
NICHOLAS J. BOURAS INC.
P.O. BOX 662, 475 SPRINGFIELD AVE.,
SUMMIT, NEW JERSEY 07901 (201) 277-1617

UNITED STEEL DECK, INC

ASSOCIATE MEMBER

MEMBER

Computer Data Base FOR STRUCTURAL SHAPES



In a continuing effort to provide design aids to structural steel designers, the American Institute of Steel Construction has developed a new Computer Data Base for properties and dimensions of structural steel shapes, corresponding to data published in Part 1 of the 8th Edition, AISC *Manual of Steel Construction*.

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   e. American Standard Channels (C)
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With LSM deck forms, you can reduce bridge construction costs by using fewer girders. A bridge that required five girders to keep the spacing short might now require only three girders. The tradeoff is deeper girders, of course, but you gain simpler construction at lower costs.

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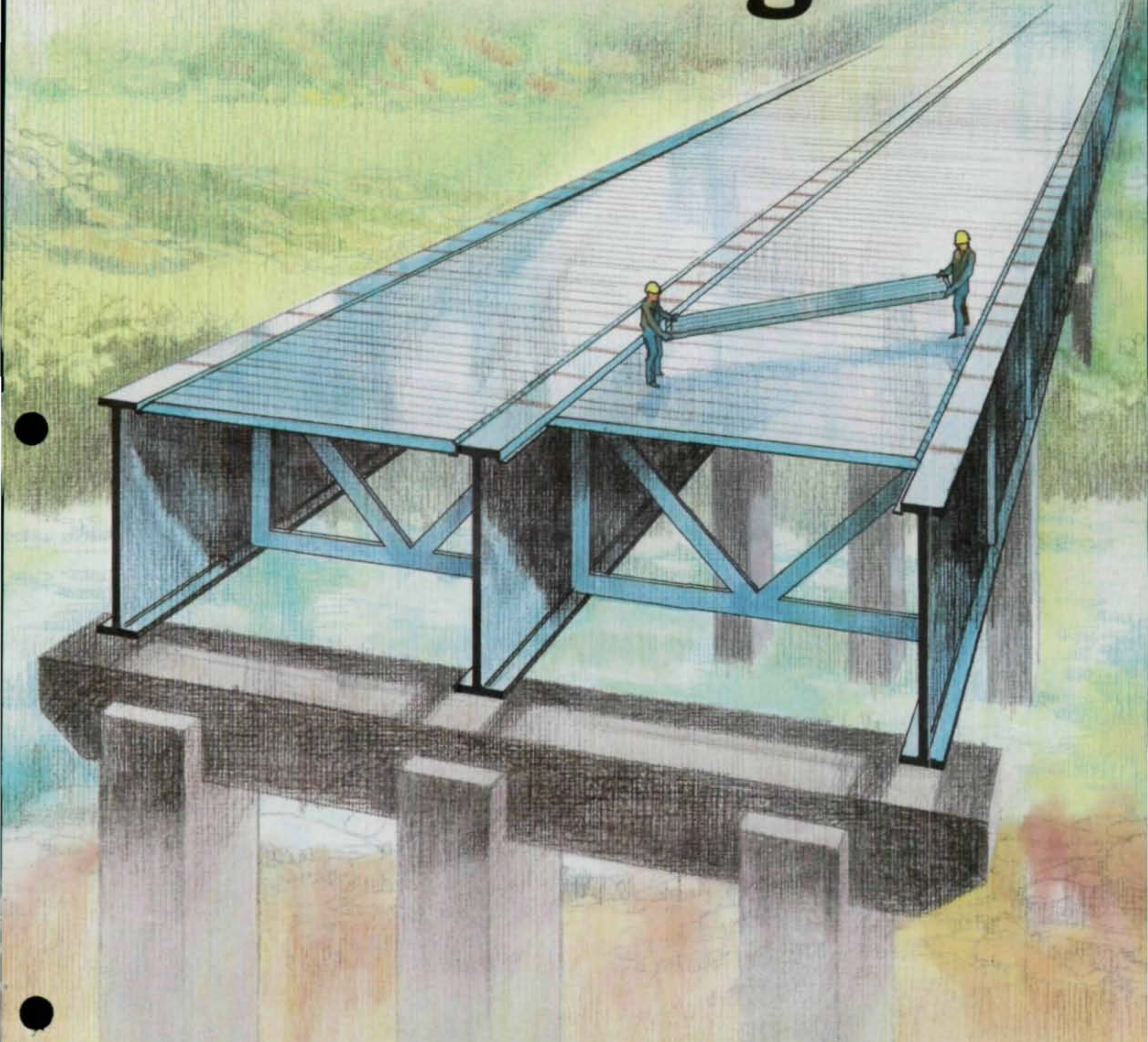

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VOLUME XXVI NUMBER 3/THIRD QUARTER 1986

MODERN STEEL CONSTRUCTION

American Institute of Steel Construction, Inc.

The Wrigley Building
400 North Michigan Avenue
Chicago, Illinois 60611-4185

CONTENTS

THREE 1986 AISC SCHOLARSHIP WINNERS ANNOUNCED

AISC named winners of three undergraduate scholarships offered this year. The scholarships, funded by three different organizations through AISC's Education Foundation, were offered to full-time, fourth-year civil or architectural engineering students in different areas of the U.S. Philip Johnson of Michigan technological University won a $2,000 scholarship, which was funded by the Great Lakes Fabricators & Erectors Association. Stupp Bros. Bridge & Iron Company sponsored a $5,000 award to Philip J. Voegtle, Jr. from Colorado State University. And Gary T. Kowatch of the University of Pittsburgh was awarded a $5,000 scholarship provided by U.S. Steel Corporation.

The scholarships, granted on the basis of academic achievement and faculty recommendation, are intended to encourage greater interest in structural steel design at the undergraduate level.

EIGHT SCHOLARSHIPS TO BE AWARDED IN 1987

Eight $5,000 graduate fellowships will again be awarded by AISC in 1987 to graduate civil or architectural engineering students who study towards an advanced degree related to structural steel design. The Fellowship Awards are granted on the basis of the candidate's proposed course of study, scholastic achievement and faculty recommendation. Applications will be available this fall at college civil or architectural engineering departments, or from the AISC Education Foundation, 400 N. Michigan, Chicago, IL 60611.

NOMINATIONS INVITED FOR 1987 T.R. HIGGINS LECTURESHIP AWARD

Applications will again be accepted this fall for the 1987 Theodore R. Higgins Lectureship Award, which recognizes the author of the most significant engineering paper related to steel in the five-year period from Jan. 1, 1981 to Jan. 1, 1986.

The winner, who receives a $4,000 cash award, presents his paper on six occasions during 1987. A jury of six distinguished engineers from the fields of design, education and the fabricated structural steel industry selects the winning author. Nominations, which should be directed to the Committee on Education, AISC, 400 N. Michigan, Chicago, IL 60611, must be received by mid-November 1986.

Northwestern Atrium: Steel Adapts to Complex Geometry

by Charles Thornton, Udom Hungspruke, Joseph Lieber, Robert DeScenza and Rainer Schildknecht

Charles Thornton, Ph.D., P.E. is president; *Udom Hungspruke* is vice president, *Joseph Lieber*, P.E. and *Robert DeScenza*, P.E. are project engineers in the structural engineering firm of Lev Zetlin Associates, Inc., New York, New York.

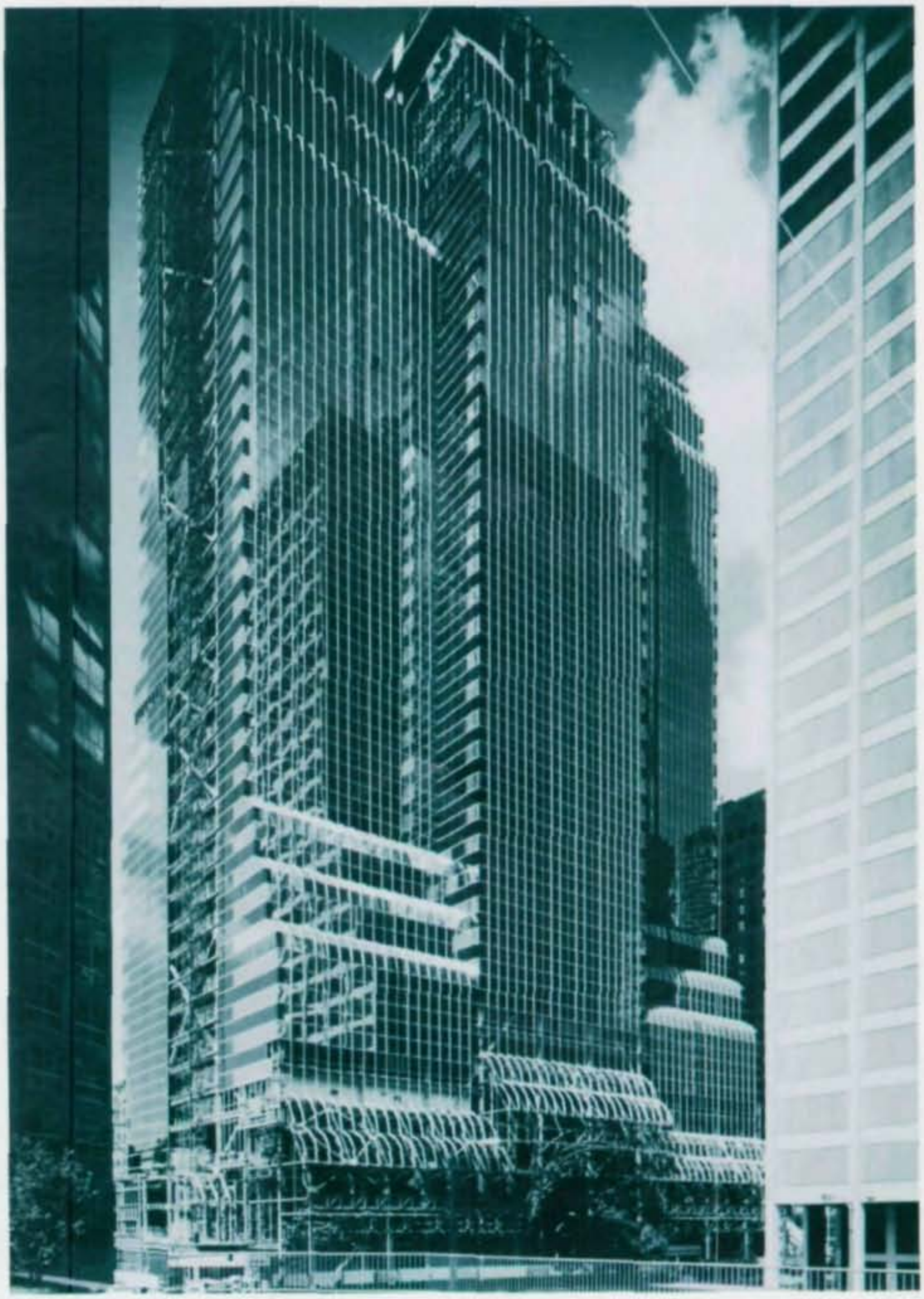
Rainer Schildknecht, AIA, is vice president of the architectural firm of Murphy/Jahn, Chicago, Illinois.

The architectural design of buildings during the last decade has evolved from box-type buildings to more exciting shapes with different geometries, contours and shapes. The Northwestern Atrium Center is a building which has benefited from the evolution. It has an indentation, a projection and roll backs (curves) in its glass facade, which create a visually exciting structure. In spite of the unique and unusual exterior appearance, the structure remained quite simple and efficient.

Architectural Concepts

The Center, located at the old site of the Northwestern Railroad Terminal just outside the Loop in Chicago, serves commuters from Chicago's myriad suburbs. One of the architectural challenges was to design an attractive building that, while it serves both as a commuter terminal and commercial office complex, would enjoy a construction method to avoid interruption to any railroad service. To maintain commuter services during the construction period, a temporary stair and bridge connecting into the existing commuter bridge and by-passing the old terminal were built prior to demolishing the old terminal.

The 40-story building built over the new railroad terminal contains about 1.7 million sq ft of gross floor space. Floor plates range from 50,040 sq ft (180 ft by 280 ft) at lower floors to 28,000 sq ft (100 ft by 280 ft) at upper floors. At the lower levels, several atria provide large, public areas. The floor plan, with its indentation at the north face and its projection on the south face, affords twice as many corner offices as the typical rectangular building could offer.



On our cover: *Chicago's striking new Northwestern Atrium Center nears completion. Cover photo by Jim Steinkamp.*

Angulated truss support and 13ft-5 in. deep truss supporting 90-ft wide indentation

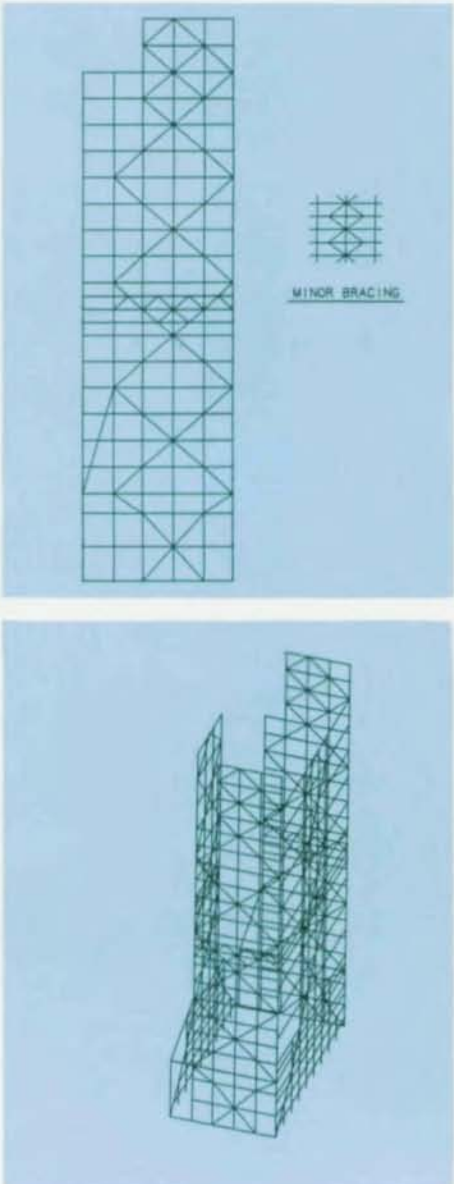
Structural Concepts Explored

Structural steel framing was chosen for the project because of its adaptability and its ability to handle the complexities of the building and its long atrium trusses. The majority of the office space is framed on a 30-ft × 30-ft grid using composite Gr. 50 W21 girders and composite Gr. 50 W16 filler beams. The structural slab is composed of 2-in. galvanized composite decking with 2½-in. of stone concrete topping. Only the lowest office floor above the retail spaces employs cellular electrified deck for access convenience.

Several wind-resisting systems were studied. A braced-core system was not feasible because the architectural requirements would not tolerate any diagonal bracing at the center of the public spaces within the atria at the base. A framed-tube system was ruled out because of the effect the indentation and projection of the plan shape of the facade had on its efficiency. Moment frames were proven uneconomical because of the height of the building, which required heavy beams and large cross-sectional area columns to develop the required stiffness and to resist bending stresses.

A major exterior bracing (superdiagonals) scheme on four faces supplemented by interior minor secondary bracing between vertical superdiagonal intersections (panel points) was finally selected (Figs. 1 & 2). Unlike most exterior braced-frame approaches, the planar elements in each face could not meet at the four corners because of their positioning in plan. This required special attention to uplift forces on corner columns. This scheme was proven to be the most economical; the building has an average of 17.5 lbs. of steel per sq ft of floor area. This scheme permitted the architect total freedom in designing all interior spaces.

All major bracing is located along exterior walls from the lower levels up. At the lower levels, continuations of these bracings are then incorporated into the interior curtain walls. All minor interior secondary bracing is located between elevator banks and terminated before reaching the public spaces below. As a result of ingenious design and close collaboration with the structural engineer, not only the wind bracing but also all tower columns are brought down to the foundations. This eliminates all pick-ups or transfers of major columns and achieves further economies in the steel weight for the total project.



Figures 1 & 2. Wind bracing at east and west elevations. Major exterior bracing, a super diagonal scheme, is supplemented by interior secondary bracing.

Cost Savings of Foundation Design

Based on site conditions and characteristics of the structural framing, a drilled caisson foundation was determined to be the most economically feasible approach. Some of the 170 caissons remaining from the existing terminal building are incorporated into the new foundation system. The bottom of new caissons are situated in either hard pan material or dense silt with a bearing capacity of 25 kips per sq ft and 35 kips per sq ft respectively. Rather than using a single, large caisson at heavy column locations which requires large machinery and additional cost, designers chose to use clusters of two or three caissons. The cluster caissons were not only proven to be more economical, but also they allowed flexibility in locating the new caissons at strategic locations to avoid interference with existing abandoned caissons which could not be removed. To achieve further economies, the existing foundation walls of the old terminal are also reused, resulting in the elimination of the earth retention system.

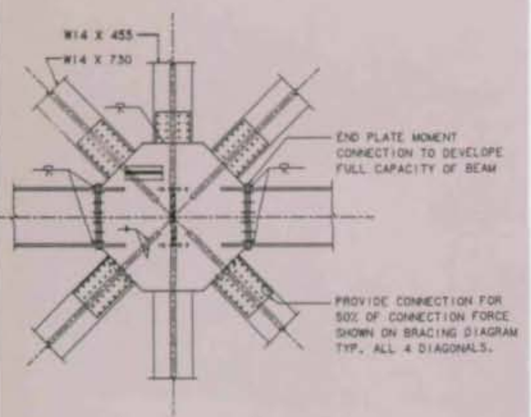
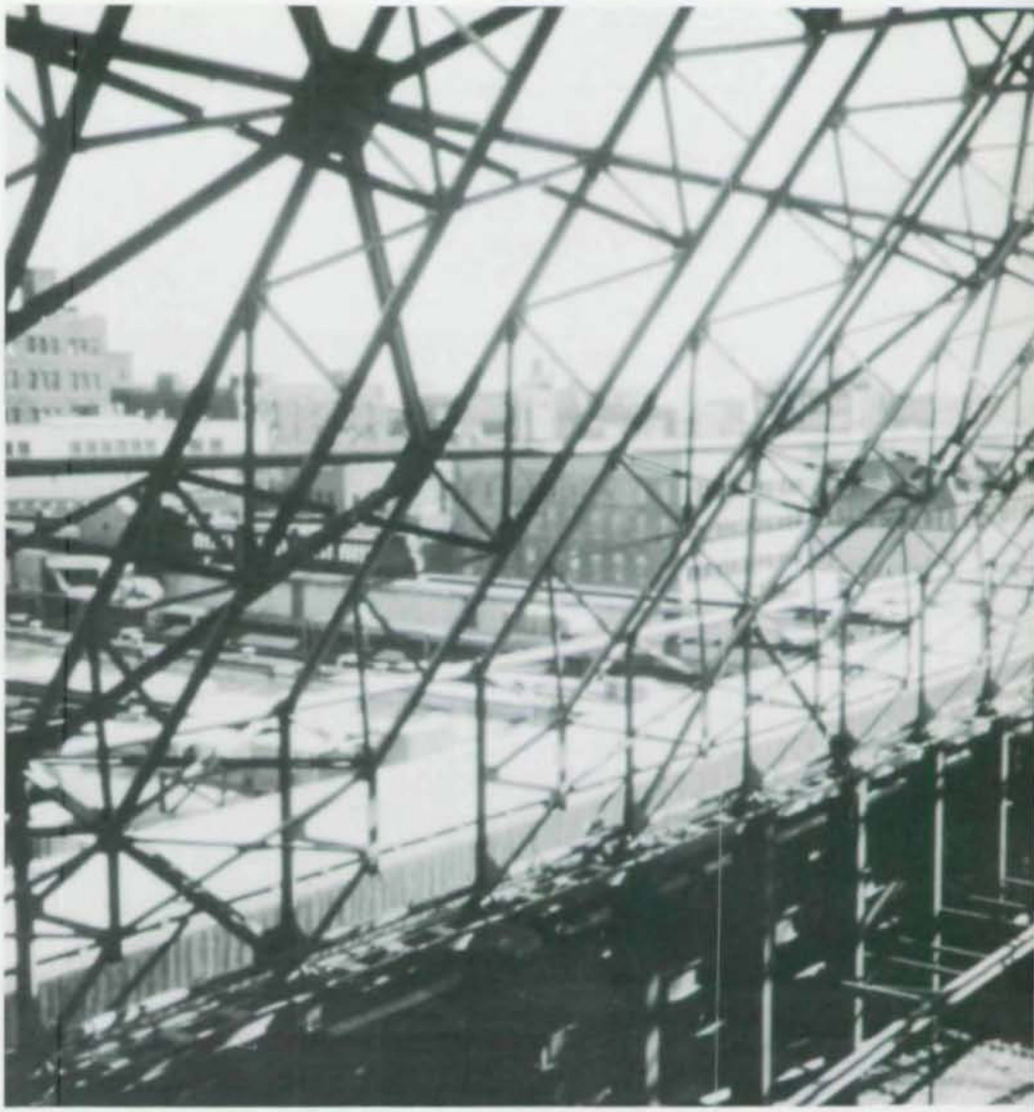
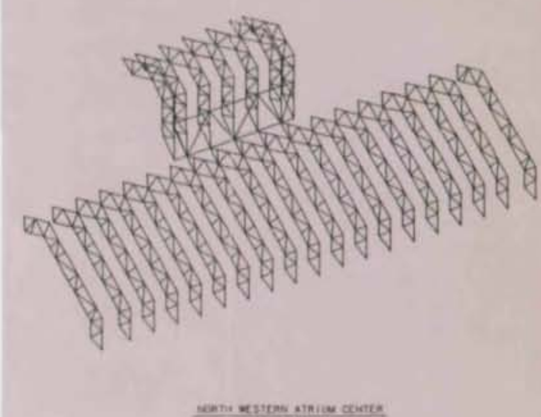


Figure 3. Wind bracing connection details





Figures 4 & 5. Angulated trusses in north atrium supported by vertical W36 columns.

Wind Bracing System

The wind bracing system consists of major exterior "X" bracing (superdiagonals) and minor interior bracing between major panel points (see Fig. 3 for connection details). The north-south direction of the major bracing has panel points every two stories, while the east-west direction has panel points every three stories. The minor interior bracing is then introduced to transfer wind loads from the intermediate floors to the major panel points of the exterior bracing system and to reduce the unsupported length of the typical columns. The major exterior bracing system was analyzed as a planar structure. Since this type of bracing tends to share significant amounts of gravity loads with the vertical columns, a separate loading condition was considered to determine the gravity loads in the bracing members. The two loading conditions, wind and gravity, were then combined in designing the major bracing members.

The minor interior bracing system was analyzed as a vertical truss spanning between major panel points of the exterior bracing. Since one of the purposes of the minor bracing is to laterally brace the gravity columns at the intermediate floors, a study was made to determine the optimum orientation of the web of the columns in order to reduce the stiffness required from the vertical truss. As a result, all gravity columns are oriented with their webs in the east-west direction. The minor bracing was then analyzed and designed for both required stiffness and for wind load stresses.

In both the north-south and east-west directions, a panel point of the major bracing was initially located at the 18th floor. However, the 18th floor is a partial mechanical mezzanine and the floor diaphragm does not reach the exterior bracing in either direction. The pattern of the bracing was altered to shift the panel point to the 17th and 19th floors. A similar situation occurred at the 2nd and 3rd floors. At these floors, one bay (across the entire width of building) of the slab at the south side was eliminated to create the south atrium. A horizontal truss was introduced across the open space at the 3rd floor to transfer loads to the major exterior bracing from the floor system.

Atrium Structures

There are two large atria, one on the north side and the south side. Both run across the entire length (280 ft) of the building. Seventy-five planar steel trusses were assembled in the atria to create the desired architectural design features. The north atrium starts at the second floor and continues vertically up to the ninth floor. It encompasses over 18,000 sq ft of floor area.

From the second floor up to the sixth floor, the north atrium, built as an appendage to the building, is enclosed on all sides by glass curtain wall. At this location, the atrium cascades out from the building. Between the sixth and ninth floors it is inset into a 90-ft wide indentation in the building. This indentation, 30-ft deep, continues up to the 30th floor.

The south atrium contains arch trusses which are the main entrance to the building. It starts at the 1st floor and continues vertically up to the sixth floor. The south atrium varies in width from 20 to 78 ft. The entire support structure, built as an appendage to the building structure, is enclosed on all sides with glass curtain wall. The curtain wall is connected to the 8-in. dia. pipe purlins. These purlins span 15 ft between trusses and are connected to outer chords of the trusses.

North Atrium Support Structure

The support structure for the north atrium may be considered as two separate sections. The lower section, which runs from the second through sixth floors of the building, is supported by a series of angulated ("broken-back") trusses 15 ft o.c. The angulated trusses are 6-ft deep, with a total overall length of approximately 65 ft (Figs. 4, 5). These trusses are supported at their base by a series of vertical W36 columns tied together across the building and laterally braced at ends by horizontal trusses.

The angulated trusses are supported on the sixth floor by either cantilevered beams or at the 90-ft wide indentation, a 13 ft-5 in. deep truss which spans the 90-ft wide indentation. To resist wind loading on the surface, the end walls of the lower part of the north atrium have 3-ft deep vertical trusses spaced 10 ft o.c. These vertical trusses are hung from the end angulated truss. The lower support of these vertical trusses is provided by the third floor; the horizontal trusses which were used to

Wind bracing line at 17th floor in building's north face brace the vertical W36 support columns also laterally support the vertical end wall trusses.

The upper part of the north atrium support structure, built into the 90-ft wide indentation in the building, extends from the sixth to the ninth floors. This part is comprised of 6-ft deep angulated trusses that span from the ninth floor down to the 90-ft long truss at the sixth floor. These trusses are spaced 15 ft o.c. as well. Lateral support is provided to the angulated and vertical end wall truss chords by cross bridging at every other panel point. A series of horizontal trusses that follow the planes of the top chords of the angulated trusses are provided for stability.

South Atrium Support Structure

The main support of the south atrium consists of two trusses spaced 17 ft-4 in. apart (Figs. 6, 7). The main trusses have an arched section over the main entrance with a flat section on either side of the arch. The outer truss has an arch with a 46-ft radius, while the inner truss has an arch with a 29-ft radius and both trusses have supports on the flat section every 30 ft.

Lateral support is provided to the main truss by perpendicular trusses at 15-ft intervals (Fig. 8). These perpendicular trusses, which slope and connect into the building at the second, fourth and sixth floors, are in turn stabilized with cross bridging. Eight radial trusses connect the two main trusses at the arch at 20° increments. A truss, also provided, for stability purposes, follows the planes of the outer chords of the sloping trusses. These range in depth from 4 ft to 5 ft. A gridwork of 8-in. dia. pipes and 6"×3" tubes supports the curtainwall at the east and west ends. In addition, five curved 8-in. dia. pipes support the facade under the arched entranceway.

Atrium/Superstructure Interaction

Due to anticipated movements in the building caused by column shortening, wind loads and thermal variations, special consideration was given in both supporting the trusses and curtain wall and in allowing for expansion joints.

The north atrium is particularly prone to these problems because the shape of the angulated trusses causes it to act as an outrigger off the building and pick up lateral and vertical forces due to building movement. The ninth floor of the structure has an anticipated vertical movement of ¾ in. due to column shortening under full dead and live load; and a horizontal movement of 1½ in. in each direction due to deflections in the wind bracing system. An analysis imposed these displacements on the angulated trusses. The forces from this analysis were combined with forces caused by superimposed loads on the trusses and those produced by differential thermal changes. The results of these forces were used to design the members.

Anticipated rotations at the top and bottom of the angulated trusses were handled by detailing these connections as pin connections. Lubrite bushing bearings with impregnated lubricant and solid stainless steel pins were employed to allow for these rotations (Fig. 9).

Deflections of the end angulated truss on the north atrium support structure was also a major concern. A slip connection was detailed to allow for vertical deflection and rotation at the bottom connections of the end wall trusses. This required placement of an expansion joint in the curtainwall along the base of the end walls. Because of the slip connection required at the base, these trusses are hung from the end angulated truss.

To deal with the differential thermal changes over the 280-ft length of the structure, four expansion joints were introduced into the north atrium support structure and curtainwall. Cross bridging was interrupted at the expansion joints and the curtain wall support purlins were fitted with a sleeve and Lubrite bushing connection to allow for movement.

In the south atrium the flexibility of the arch trusses was analyzed and it was determined that no expansion joints were necessary in the east-west direction. □

Architect
Murphy/Jahn
Chicago, Illinois

Structural Engineer
Lev Zetlin Associates, Inc.
New York, New York

Construction Consultant
M. E. Oppenheim Associates
Chicago, Illinois

General Contractor (joint venture)
Gust K. Newberg Construction Company, and Paschen Contractors, Inc.
Chicago, Illinois

Steel Erector
American Bridge Division, U.S. Steel Corp.
Chicago, Illinois

Owner
Northwestern—Atrium Center Associates
Chicago, Illinois

Developer
Tishman Midwest Management Corp.
Chicago, Illinois

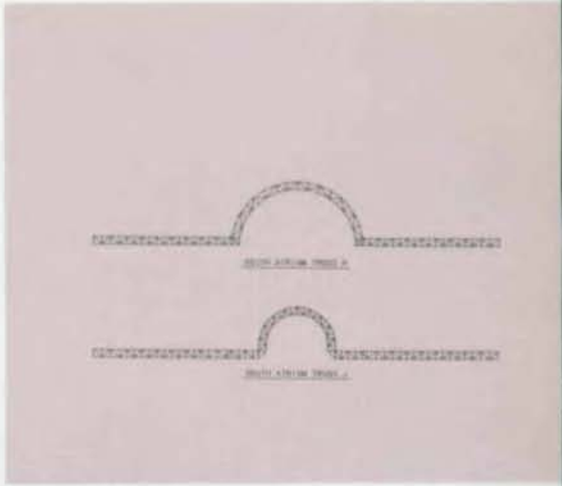


Figure 7. Main support of south atrium

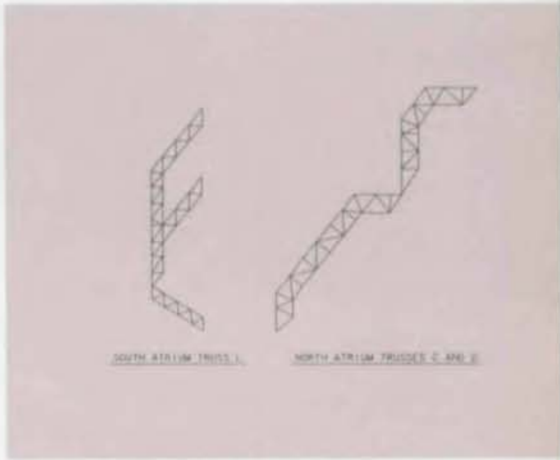


Figure 8. Lateral support to main truss

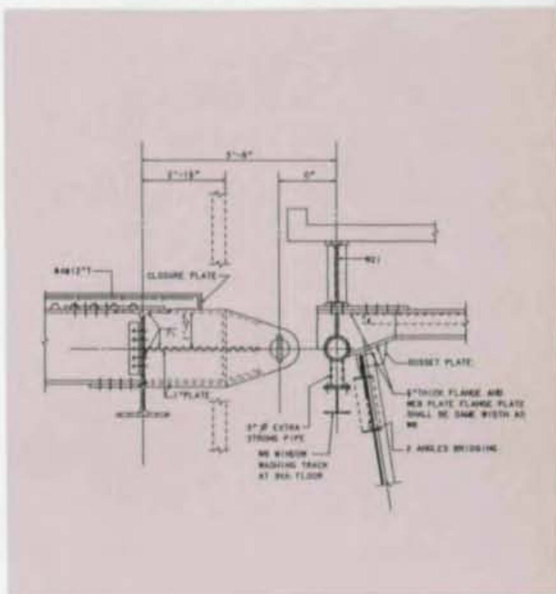


Figure 9. Detail at top of trusses

Composite photos of Bradley Airport show extensive addition to congested airport facilities. Steel framing accommodated design, sped construction.

Bradley International: A Unique Design Experience

by Chan K. Lin and Robert Lie

In 1980, the State of Connecticut concluded the existing Murphy Terminal at Bradley Airport needed expansion to relieve the congestion and bring the airport up to the International standards. The state retained an architectural team to design a new addition. However, prior to the actual design of the structure, the team requested involvement in the update of the master plan. Their involvement resulted in a new development concept which included a strategy to expand the facility and remove existing, outdated structures. The concept then went to the state for review.

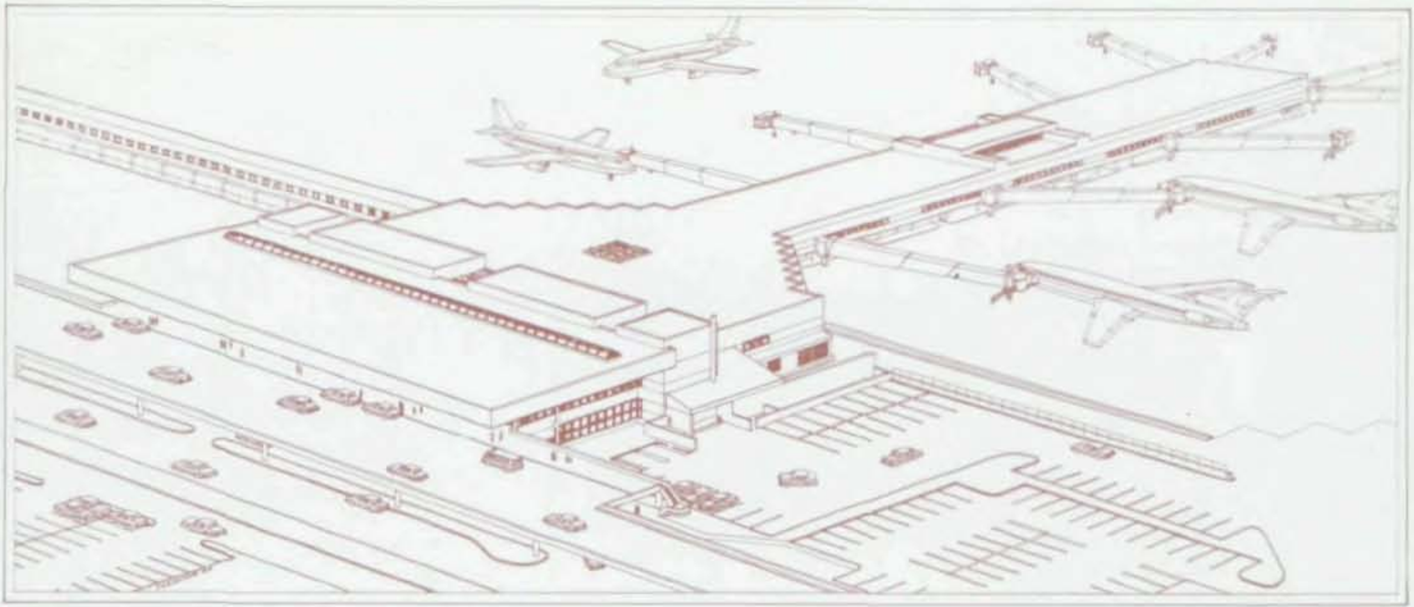
Architectural Concept

The architectural concept of the new addition was to create an image of a structure to reflect the state-of-the-art transportation mode it would serve. The structure employed metal skin and windows to create an appearance that blends well with surroundings and creates a system adaptable to change. Its interior layout followed a circulation pattern to minimize travel distance for the passengers both within the building or to adjacent terminals and concourses.

The new two-story building serves four airlines, plus offices and concession shops. The terminal is dominated by a spacious ticket lobby and restaurant areas. The openness of the lobby is accentuated by a very light and open air design contributed by a large linear skylight and the all-glass exterior wall. The main stairway and escalator is also very brightly lit by the cluster skylight at the roof. The terminal boasts all the modern amenities such as a canopied enplaning area by the cantilevering roof and curb-side baggage check-in facilities.

Beyond the ticket lobby, the terminal houses various concessions in an area which serves as the focal point for passenger circulation. This area is also made very

Chan K. Lin is president, and Robert Lie a project engineer, for the consulting engineering firm of Lin Associates, Inc., Boston, Massachusetts.



Rendering of new airport facilities details added facilities.

bright by a cluster skylight above. The transition area between the wide terminal and the narrower concourse, described during the design phase as the "saw-tooth area," will be mainly occupied by a 4,300-sq ft restaurant and a 3,700-sq ft lounge. Curved glass windows maintain the airy feeling throughout. The concourse area is designed for maximum airline positions and plenty of space for boarding passengers.

Structural Steel Framing for Flexibility

From early design stage, structural steel was chosen as framing for the superstructure, mainly for reasons of architectural need, shorter erection time and its inherent flexibility for future horizontal expansion. First, the versatility of the structural steel accommodated the requirements of the metal skin exteriors in creating a modern image of the structure. The complex system of curved exterior glass walls and metal skin is easily supported by the steel framing.

Approval of the conceptual design mandated a tight construction schedule which the architect had to implement. A fast-track construction for the foundation was chosen to shorten the time required to construct the building. Structural steel scored very well in terms of short erection time in comparison with other modes of framing. Finally, the new master plan for the airport called for provisions to facilitate future expansions. Structural steel was a logical choice for this conceptual criteria.

The overall framing system was comprised of reinforced concrete foundation, slab on grade and in the mechanical basement area, and structural steel framing for the superstructure. To meet the occupancy date, the entire final design and production of structural contract drawings for the superstructure were completed in less than four months. Fabrication of the structural steel commenced immediately.

The superstructure is a combination of a truss system at the front terminal roof, rigid frames for the concourse area and composite steel beams and columns for the remainder of the building. For the main floor and roof slab, $3\frac{1}{4}$-in. and $2\frac{1}{2}$-in. thick structural lightweight concrete were used respectively over 2-in. deep, 20 ga. composite metal deck to meet both strength and fire-rating requirements. Typical floor and roof beams were full composite members with the metal deck and $\frac{3}{4}$-in. dia. headed-sheer studs welded on. The full composite system was chosen to gain a higher efficiency in steel usage and its rigidity against deflection over the noncomposite or partial composite design.

Terminal Area Meets Architectural Demands

The front entrance and lobby portion of the terminal roof was framed using steel trusses spaced 25 ft o.c. as main members with 20-in. deep open-web joists between. The framing was chosen to meet an architectural feature requiring a long cantilever canopy at the front entrance to protect passengers during inclement weather. The design also called for a 20-ft cantilever canopy on either side of the terminal. Typical main trusses were approximately 7-ft deep, with structural tees used for top and bottom chord and double-angle members for web elements. These trusses were oriented longitudinally to the building with a 31-ft cantilever at the front and span over two bays about 75-ft long at the back. The tips of the cantilevers were braced horizontally by trusses at the bottom chord level made of 12-in. channels and angles. To achieve the cantilever at each side of the terminal, two transverse trusses were used to pick up the edge truss at their overhanging end, thus creating a double cantilever effect for the edge truss. Since two end columns were offset about 5-ft away from column grid to clear a way for the entrance, the two transverse trusses are also supporting the first interior longitudinal truss at their interior span. Inverted K-bracings between the transverse trusses provide for lateral rigidity.

Connections for the structure, in general, were welded for shop connections and bolted in the field. End-plate connections were used between trusses and columns to better transfer the moment at the trusses. On the roof level, columns were braced at the top by vertical inverted K-bracings to provide for rigidity and to transfer lateral forces to the reinforced concrete stair shaft on both sides of the terminal. The shafts were designed as the primary shear-resisting elements for the terminal area in its transverse direction.

A two-dimensional computer analysis was used for the preliminary design of the roof truss system. During the final design stage, a comprehensive, three-dimensional truss computer analysis predicted how the trusses interact with each other and with the bracing system. The analysis provided valuable information not only in determining the stresses on the individual elements and ascertaining their compliance with the Building Code, but also about the critical deflection of the truss system, particularly at the tip of the double-cantilever trusses. With computer in hand, designers were able to determine the amount of camber that needed to be incorporated in the truss fabrication to overcome deflections under service load.

The restaurant, or saw-tooth area, is another challenging space that required close coordination between architect, engineer and window manufacturer. The design of the building called for a transition between the wide-bodied terminal area and a narrower concourse in the form of 10-ft square module steps; thus the saw-tooth name. These step surfaces received a curved exterior window system to provide maximum natural light penetration as well as a pleasant viewing area.

The basic roof framing systems were a series of transverse beams supported at one end by an interior longitudinal beam extending to the next module. Five-by-five-in. tubular columns from the second floor served as the other supports for beams. The exterior end of the beams cantilever 5 ft from the tubular column to provide support for the longitudinal edge beams. The framing system for the roof was repeated at the floor level, except that steel posts were replaced by deep wide-flange transfer girders spanning diagonally between first-floor columns. The second-floor framing was offset from the roof framing to account for the curved window system it received. The slightly overhanging window modules lend a pleasing, functional transition to the concourse area.

Concourse Rigid Frame

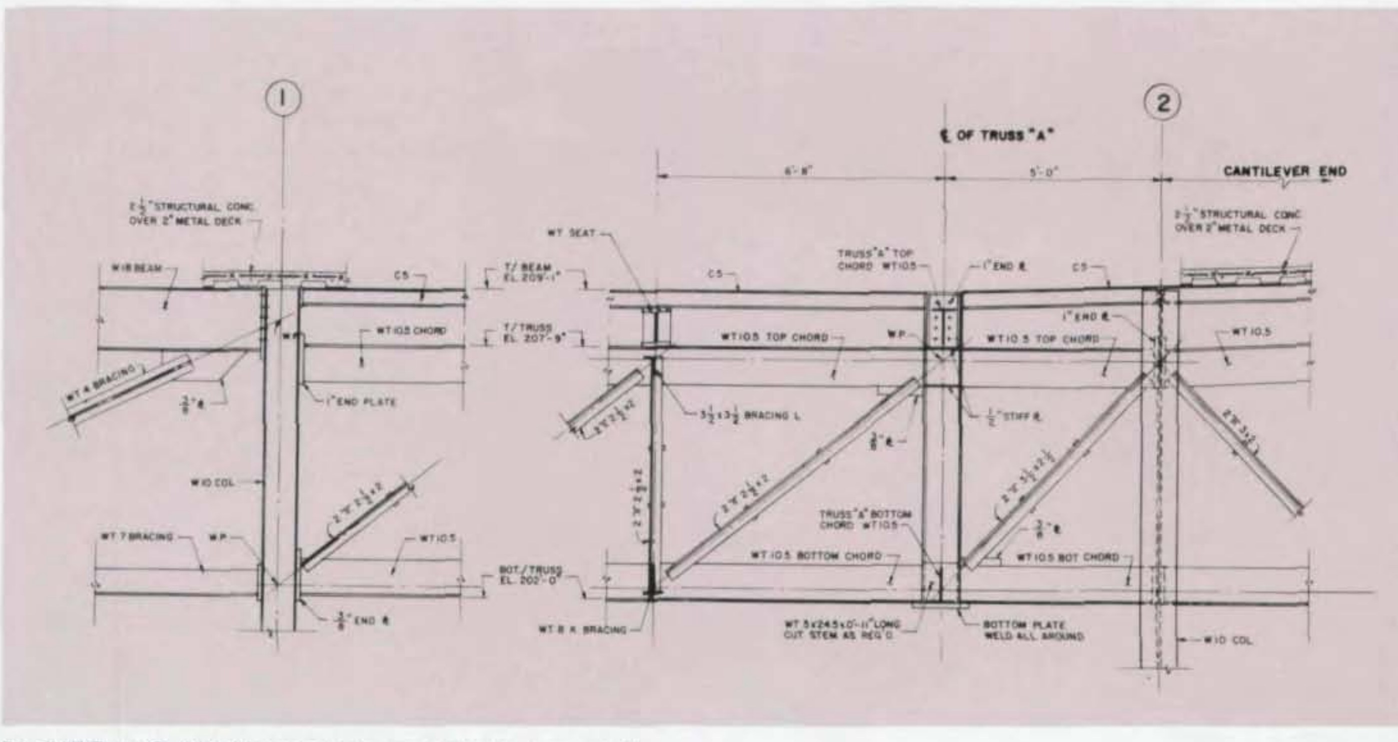
The new concourse is a 430-ft long and 75-ft wide structure. Typical framing was a 55-ft span rigid frame at the roof level with a 5-ft cantilever on each end. At the second-floor level, framing was a two span 27'-6" rigid frame with a 10-ft long cantilever on each end. The frames were spaced at typical 25-ft intervals, with W33 beam elements at the roof and W21's at the second floor. The difference in cantilever length between the roof and floor framing was to provide support for the 5-ft radius curvature at the top of the exterior window wall system. The windows were then supported by beams and outriggers spanning the rigid frames.

The rigid-frame modules at the concourse were interrupted halfway by a 95-ft long stair system and mechanical penthouse. In this area, the basic framing at the second floor was used, except around the stair openings. In the roof, however, two floor-height parallel chord trusses serve as a main support system for the mechanical penthouse floor and roof. Beams framed into these trusses pick up the slabs. Expansion joints on both ends of this structure free it from thermal movements of the concourse.

Construction

Construction of this $20-million project proceeded very well. In particular, very few structural problems were encountered. As mentioned earlier, to save time, as well as to eliminate pouring concrete during cold weather, a fast-track construction was chosen for the foundations. The method was used in spite of the obvious design risks inherent with fast tracking.

Steel fabrication was performed under a very tight schedule to meet the targeted erection date. An experienced erection crew, working closely with the general contractor and the construction manager,



Detail "I"-Truss C. All field connections were bolted (unless noted).

made it possible to erect the total 1,060 tons of steel in only 2½ months during frigid mid-winter weather. Erection of the truss system for the roof of the terminal area proved a challenge for all. Much effort went into fine-tuning the trusses, especially at the two-way or double-cantilever areas, to bring the system to the final camber specified.

Torque Control Bolts

To facilitate construction, the contractor opted to use high-strength ASTM A325 torque-control or load-indicating bolts to field-erect connections and standard A325 hex-head bolts for shop connections. Using these bolts provides a speedier connection inspection, provided their limitations are recognized. The round-headed configuration of these bolts negated using standard torque-test instruments. To overcome this, field engineers came up with a testing system that used a tensioning device on bolts at random for every batch in the field, in combination with on-site, spot torque testing of bolts in place. The torque-control bolts were also found not very suitable for connections that required gradual tightening. This occurred primarily at the trusses, where, to achieve the final erection cambers specified, gradual tightening and proper adjustments were required for the column connections and between trusses. Therefore, for the truss connections, standard A325 hex-head bolts were used.

Conclusion

Design of the new terminal and concourse C addition incorporated a carefully planned strategy that allows for future expansion as well as meets current transportation needs. Structural steel was used as framing elements to satisfy functional requirements and the aesthetics defined by the architectural features incorporated in the design. The innovative use of the various structural steel framing systems in fulfilling the design requirements made the building a unique design and construction experience. □

Architects
CE Maguire, Inc.
New Britain, Connecticut and
Reynolds, Smith and Hills
Jacksonville, Florida

Structural Engineer
Lin Associates, Inc.
Boston, Massachusetts

General Contractors
Shamrock Construction (foundation)
New London, Connecticut
W.E. O'Neil Construction Company (superstructure)
Hartford, Connecticut

Owner
Connecticut Dept. of Transportation,
Bureau of Aeronautics

Ticor Title's new office in downtown San Diego. Designers "cut corners" to achieve striking architectural effect.

Ticor Title: "Bracing" for a New Look

When Ticor Title Insurance Co. originally selected a .3-acre corner site at Tenth Ave. and B St. in downtown San Diego, they intended to construct a 30,000 sq ft office building with a traditional, "old law-firm" design—the brick veneer concept. But when Ticor entered a joint venture development on the low-rise building with WW Investments, a contemporary concept was commissioned and square foot requirements doubled to create income-producing space—but the lot size was the same.

The project architect, Dennis Hyndman, chose steel to solve the problem. And the $5-million, six-story building has become the only local downtown building to express a steel braced-frame structure, cloaked in an expandable, stucco-like exterior wall insulation.

"The challenge was to maximize use of

Site limitations on Ticor building were overcome by architect in using diagonal braces to increase floor areas and improve visual appearance (l.). During early construction, crane dropped equipment into hole to work.

the available space, double the building size from the original concept, create a large landscaped area and an intriguing visible appearance for one of the most trafficked intersections in the city," Hyndman said. The final product gave the joint venture 67,000 sq ft of office space and 30,000 sq ft of subterranean parking for 90 cars.

The architect overcame the site size limitations by using diagonal support braces to increase floor areas above ground level, forming a distinctive "X" design on the visible sides of the exterior. The braced-frame system uses less steel than most office buildings, frees up the corners and requires more concrete in the footings.

Cutting Corners—Architecturally

All four exterior sides of the building are load-bearing, but the publicly visible Tenth and B street sides have aesthetic configurations which required significant amounts of "tuning" to insure consistent stiffness in the braced-frame system. The architectural concept also resulted in a Rubik's Cube inverted entryway, incorporating the steel diagonals with horizontal floor beams to form trusses, which cantilever from two centered columns on each side out to the corners.

"The diagonals are load-carrying, so the use of steel enabled us to eliminate corner columns and use more floor space for Ticor to occupy or lease," said Hyndman. Ticor occupies 40,000 sq ft, and the joint venture plans to lease the balance.

The challenge of the space-conscious design affected construction. Bolted erection connections required for the complicated bracing systems were constructed to allow for on-site assembly without shoring, and to avoid interference with the exposed bracing. The column and bracing assemblies were detailed and prefabricated to demanding tolerances, then trucked to the site.

High-strength (ASTM A449) steel-embedded anchors were used at the base of the Tenth Avenue and B Street frames, however, the south and west sides incorporated No. 18 rebar into the base of the wall columns. Standard (ASTM A36) steel was used for the bracing and tension members. A total of 690 tons of steel went into the structure.

The striking, inverted entryway is framed with the upper floors stepped back from the exterior wall lines to create interior space at the ground floor. This unique look required the corners of the building to literally be erected from the top down. Since much of the structure was constructed in pieces to result in a flexible, limber unit, parts were assembled on the ground then lifted into place by crane. But getting the crane on site posed another challenge.

Access to erect the building was available only by Tenth and B streets. While one street was within the limits of the construction barricades, the other required street-use permits from the city. Because the streets are so heavily traveled, only designated blocks of time were available for use of the 100-ton truck crane in the city traffic lanes.

Complex Steel Framing

Complex steel framing was required at ground level to accommodate the extensive landscaping desired, build the ground floor three feet above grade and address encroachment of below grade parking onto public property. The foundation work, specifically the need to move trucks in and out of the deep site, was challenging. During the early stages of construction, earthmoving equipment had to be placed in the hole by crane.

To achieve gravity load support and create an earthquake-resistant horizontal diaphragm, double framing was used on nearly 50% of the ground level floor; the structural floor at ground level slopes from a point about 30 ft inside the walls to a level 3-6 ft below the sidewalk on Tenth and B. A level floor constructed above this sloping frame extends to the planter walls. The composite deck system used was standard 4½-in. concrete over 3-in. metal.

The live load for the majority of the structure ranged from 20 to 70 psf. However, unusual loading conditions for the third, fourth and fifth floors, where Ticor keeps records and maps, required a 100-lb. live load.

Besides the X brace, the Ticor building features Dryvit and blue trim on the exterior, and boasts two 20x20-sq ft balconies on the fifth and sixth floors which provide views of city and harbor. Hyndman believes more downtown office projects should feature such amenities. "It's a feature more buildings in San Diego should consider having to respond to the climate we enjoy," he said.

The architect incorporated Dryvit with gray, non-reflective glass and blue mullions and trim to create a "light, fresh looking building." It offers economical insulation and made it possible to install wide floor-to-ceiling windows which are tinted but more transparent than most in modern office buildings. It is a highly energy-efficient, patented exterior wall insulation and finish system, which can be applied to new and restored buildings. The system consists of 1 - 2½ in. of expanded polystyrene (EPS) with a natural sandblast finish. "The Ticor building is the only structure in the city of San Diego to use this system," said Hyndman. "When we designed Ticor, we took all the elements of San Diego's climate into consideration, in addition to the orientation and position of the structure," he explained. "We added an extra layer of insulation on the exterior southwestern side to provide a more energy-efficient structure.

"The braced-frame concept has to be something the client is willing to live with because of the inherent blocked views for tenants with offices behind the "X" of the braced frame," Hyndman said. "But I'm very pleased with the project. It does what we wanted it to do. It's a light, airy structure at the corner of Tenth and B. The area is fairly undeveloped at this point, but we hope this building will attract other developers." □

Architect
Deems/Lewis & Partners
San Diego, California

Structural Engineer
Atkinson, Johnson & Spurrier
San Diego, California

General Contractors (joint venture)
Fred Watts Construction Co., and
Fletcher Company
San Diego, California

Steel Fabricator
Junior Steel Co.
City of Industry, California

Owners (joint venture)
Ticor Title Insurance Co., and
WW Investments
San Diego, California

Hartford's Union Station Center, a landmark restoration (south wing)

Renderings courtesy Halcyon, Ltd., Commercial Development Consultant

Hartford Union Station: Diversity of Steel is the Key

by Karen J. Heroy

Karen J. Heroy is communications director for Tai Soo Kim Associates, Hartford, Connecticut.

Hartford's Union Station Transportation Center is a landmark restoration project full of complexities. It is diverse in its mixed uses as a transit and a commercial center. Rehabilitating the brownstone building according to historic guidelines made recreating and matching materials a priority.

Steel is used throughout the building in various ways—and its importance is an understatement. Steel provides the structure for the new addition, supports the existing railroad tracks and is functional and decorative throughout the brownstone building.

The station's prominent location on the western edge of downtown Hartford is significant since it provides an image for this part of the city, essential to revitalizing this area's economic growth and development. Key to making this happen is an unusual ownership structure and a complicated funding package. The Greater Hartford Transit District owns the existing building and the new addition, while Amtrak owns the abutting railroad tracks, trestles and platforms. The district is developing the building's transit sections and a local development consultant the commercial space. The project financing includes funds from federal and state sources and from a consortium of local investors, Union Station Associates.

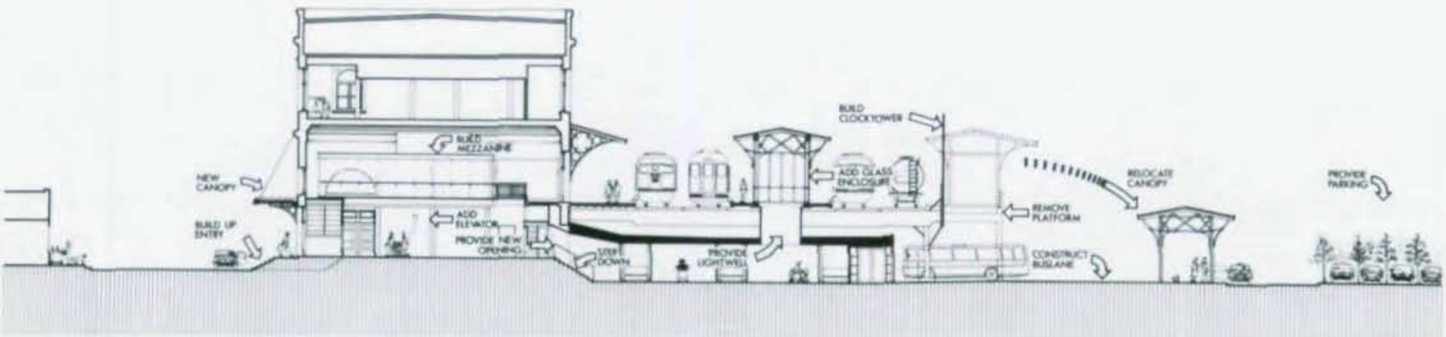
History of the Brownstone

The existing building is constructed of brownstone brought from the Portland, Conn. quarries and cut into large, rough-faced blocks. The Romanesque style building seen today was not the original structure.

In 1849, a railroad station was built with the tracks at street level. Because of numerous traffic accidents, the station was rebuilt in 1889 in its existing form, with a gabled roof and with elevated tracks. In 1914 a fire gutted the center portion and damaged other parts of the building. This reconstruction gave us the existing building with its current flat roof design in the center. For years, Union Station thrived as a bustling center owned by the New York, Hartford and New Haven RR. As rail passenger traffic decreased in the 60's, the station started its decline. By the early 70's, deterioration was fully apparent.

Design Program

In 1976, the architect was commissioned by the Greater Hartford Transit District to study the station for its potential use as a



North wing with greenhouse cafe looking toward state capital. Cutaway rendering details facilities.

transportation center. Through that study, an overall concept dividing the building into two parts developed:

1. Transit services will be located in the new addition, accommodating local and regional passengers for rail, bus, taxi and airport limousine service;
2. The brownstone building will function as the main lobby and as a commercial center with offices on the upper floors and mezzanines, with retail stores and restaurants on the ground level.

Phase 1. During this phase of design, the architect renovated and made adjustments to the rail-related parts of the project, the railroad tracks, trestles and platforms. The fundamental role steel plays in the project is apparent here. To provide clearance for the addition and for buses, the existing knee braces were removed and new equivalent braces installed at higher locations. Cross bracing was eliminated where it interfered with clearance, and added in more suitable areas. A large new plate girder was installed and the existing girder cut to make room for the new stair from the bus level to the railroad platform above.

Of the four original passenger platforms, the western-most was not being used, nor did Amtrak intend to use it in the future. To maintain visual continuity throughout the project and to use as many of the original parts as possible, this platform was dismantled and the canopy installed on the ground (see section). Located along the project's western boundary, the canopy will serve as a waiting and drop-off area for local buses, taxis and limousines outside the facility.

Phase 2. This phase entailed two major tasks: repairing the roof and cleaning the exterior brownstone. The terra cotta tiles were removed to repair the gypcrete roof deck in the north and south wings, and then both wings were completely reroofed. To maintain the roof's weathered texture, as much original tile as possible was reused and matched with new tile made by the original manufacturer. Installing skylights in the flat portions of each wing roof brings much needed light into the new office areas. Painting the exposed decorative trusses will highlight the interior rehabilitation.

Phase 3. Currently underway, in this phase the renovations to the brownstone building are being completed, adhering to historic preservation guidelines and the new addition constructed.

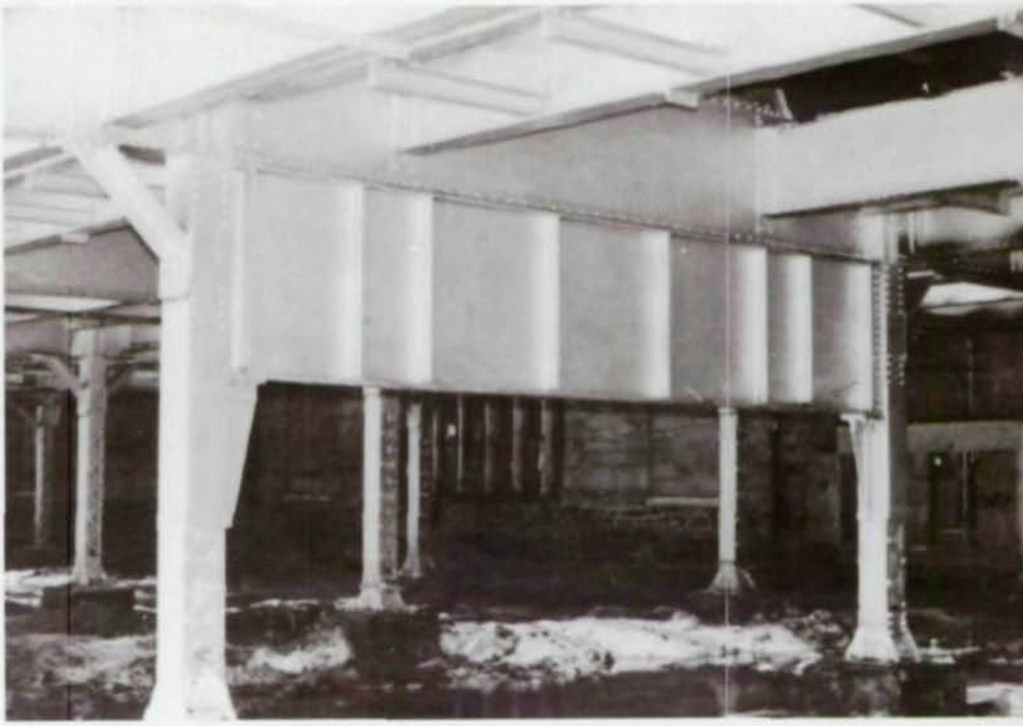
Roof. Within the roof structure, existing steel angle hangers support the third floor, rather than columns from below. This leaves the second floor virtually column-free. On the third floor, the ceiling has been cut to receive new steel for the mechanical platforms. The new steel will span the entire space, bearing on the exterior masonry walls, without putting additional load on trusses or floor. New clear-span beams also were installed throughout the second floor south wing, resulting in a column-free first floor.

Throughout the brownstone building, the versatility of steel is apparent. Such qualities as strength, durability and flexibility make steel a major ingredient in the project. The brownstone building also exemplifies an integration of the building's historic features with the new.

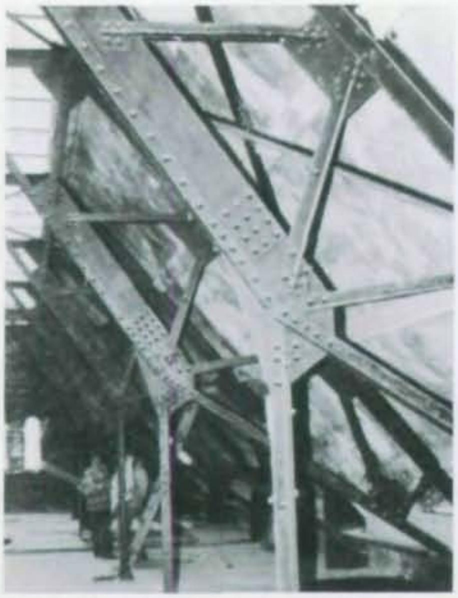
The Great Hall. In the center area of the building, the Great Hall is being modified to serve both halves of the project, functioning as a transit waiting room and a commercial space. The two new mezzanines, which will not touch the existing ceiling, will provide office space. Two new elevators will connect all three levels, with one providing access for the handicapped from waiting room to ticket lobby. The front entry and its canopy will be recreated to approximate original designs of the ornamental steel canopy and the stairs which led into the building.

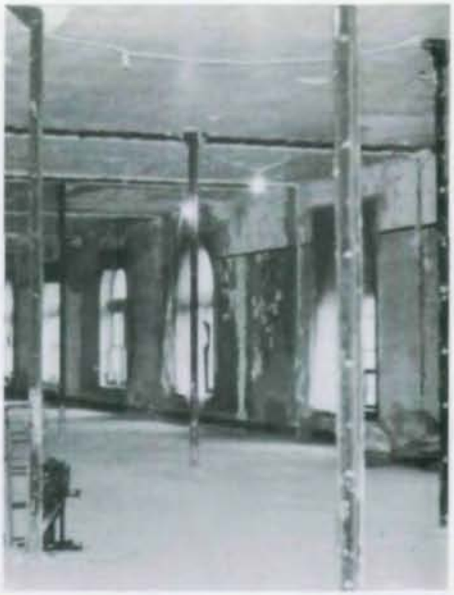
South Wing. A glassed-in pavilion restaurant is being built where the South Wing's exterior shed structure was located. The original metal brackets were measured carefully to recreate them for the new pavilion canopy. Because this restaurant is visible from a major local artery, and also because it offers a spectacular view of the Capital, it should be a significant draw to the new Center.

North Wing. A new greenhouse structure will be built adjacent to the North Wing, facing Union Place. The design combines wood decking, glass and exposed steel stuctures. Reminiscent of the original shed roof which once sheltered the

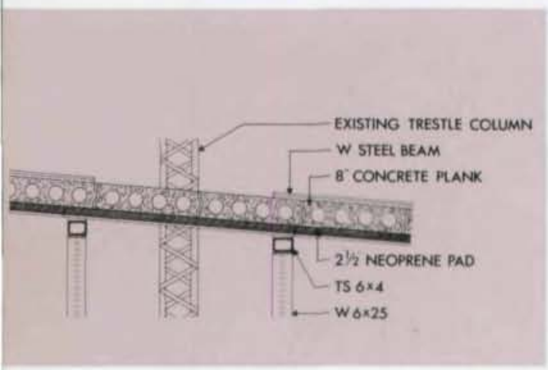


New plate girder supports trestle of remodeled depot.

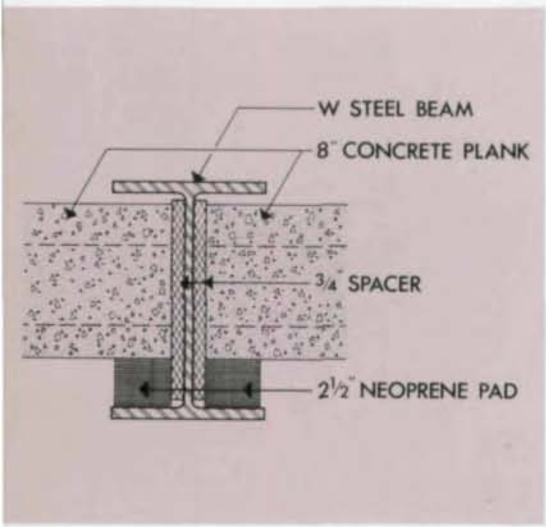




Steel trusses in 3rd floor (l.); and floor hangar supports (c.) over waiting room. Modification to existing trestle support (r.)



Sketches of trestle remodeling details



baggage handling area, this design enhances the building's historic context. The greenhouse will contain a cafe area with potential sidewalk seating. In conjunction with a proposed new sidewalk, and prohibition of parking along the building, the addition will greatly improve appearance of the area.

New Addition. The floor plan illustrates how critical space is between the roof of the new building and the existing railroad platforms. The structural concept grew out of a need to minimize the structural depth and to minimize noise transmission from overhead trains. Consequently, a precast concrete plank was selected as a suitable roof structure, supported by new steel beams and columns. Neoprene bearing pads minimize vibration from above (see plan).

One of Amtrak's structural requirements was that the existing rail-related steel structures not be touched. It will continue to be examined, maintained and owned by Amtrak. Because of these requirements, the resultant design specifies heavier steel than normally used in such areas. Because of the sloping tracks above, depth of construction in this area varies from 18 to 36 in. To achieve this tight construction, 8-in. concrete plank is supported on the bottom flanges of new W14 x 61 beams.

Future of the Project

Completion of the Transportation Center will have a significant impact on revitalizing the western edge of downtown Hartford. It serves as a gateway from the Asylum Hill neighborhood and is visible from I84. With the station rehab underway, development activity has increased along the immediate and surrounding streets with newly renovated shops and restaurants. It is expected the continuing work at Union Station will stimulate development of the remainder of the area.

The complicated ownership structure creates an economically fruitful venture, bearing benefits to all involved. Amtrak maintains its rail service in a more sophisticated environment, the Transit District gets a multi-modal transportation center, and the investors get a new and active commercial center.

After 10 years of study, design, modifying and accommodating the concerns of various owners, the design concept is finally transformed into steel and stone. The dream of a revitalized Union Station is realized! □

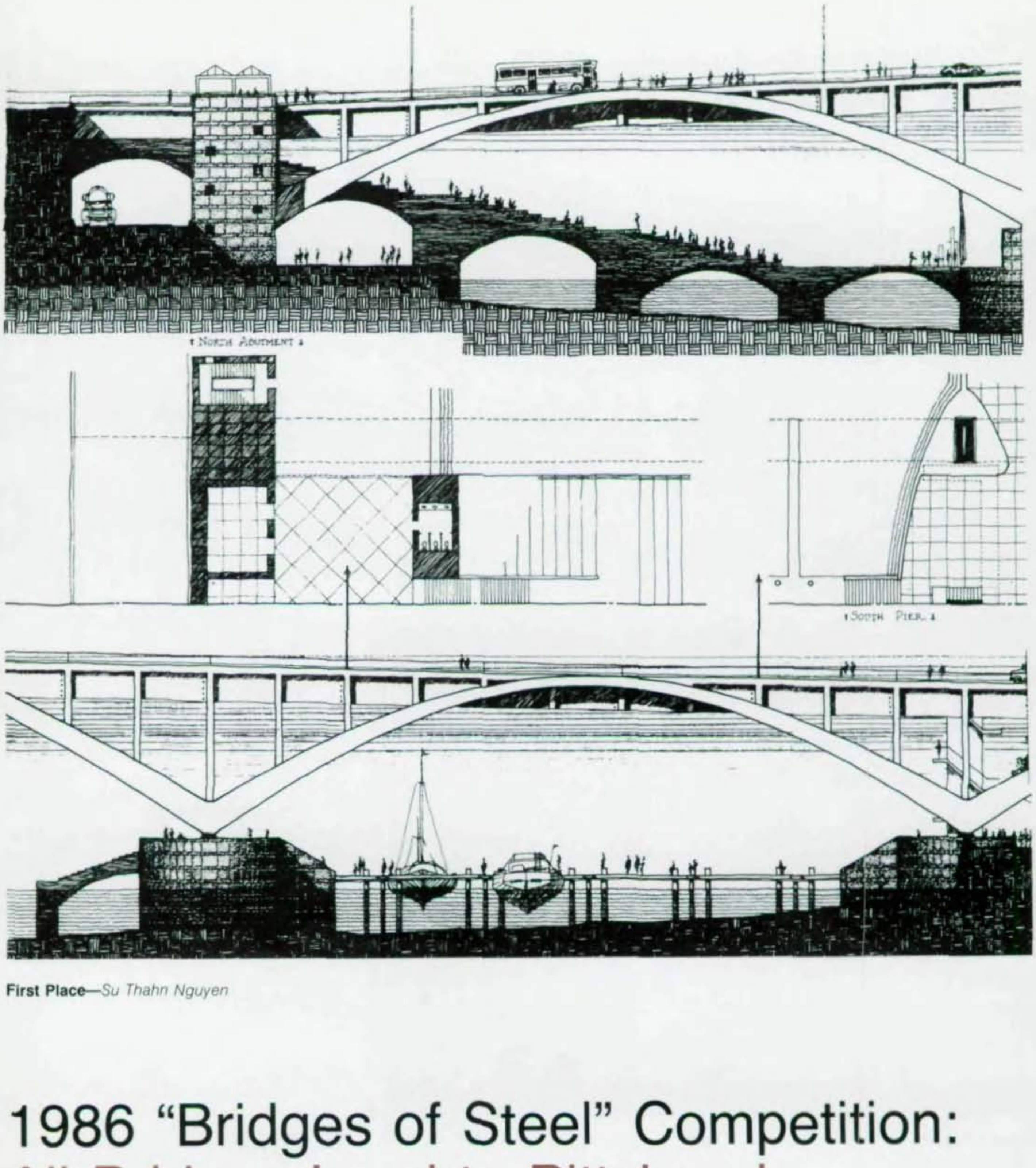
Architect
Tai Soo Kim Associates
Hartford, Connecticut

Structural Engineer
R. A. Goodell & Associates
Glastonbury, Connecticut

General Contractor
Northington Builders, Inc.
Simsbury, Connecticut

Steel Fabricator/Erector
Topper & Griggs, Inc.
Plainville, Connecticut

Owner
Greater Hartford Transit District

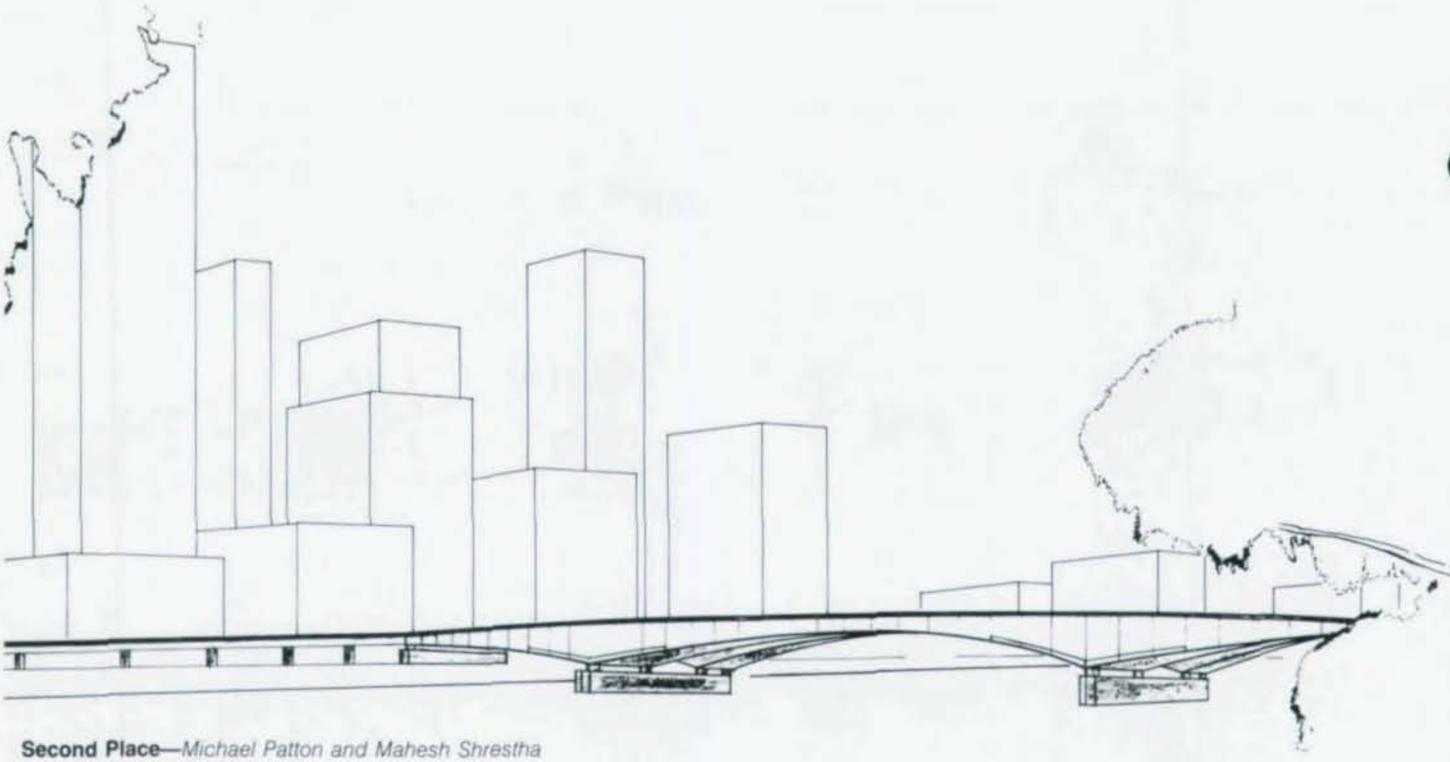


First Place—*Su Thahn Nguyen*

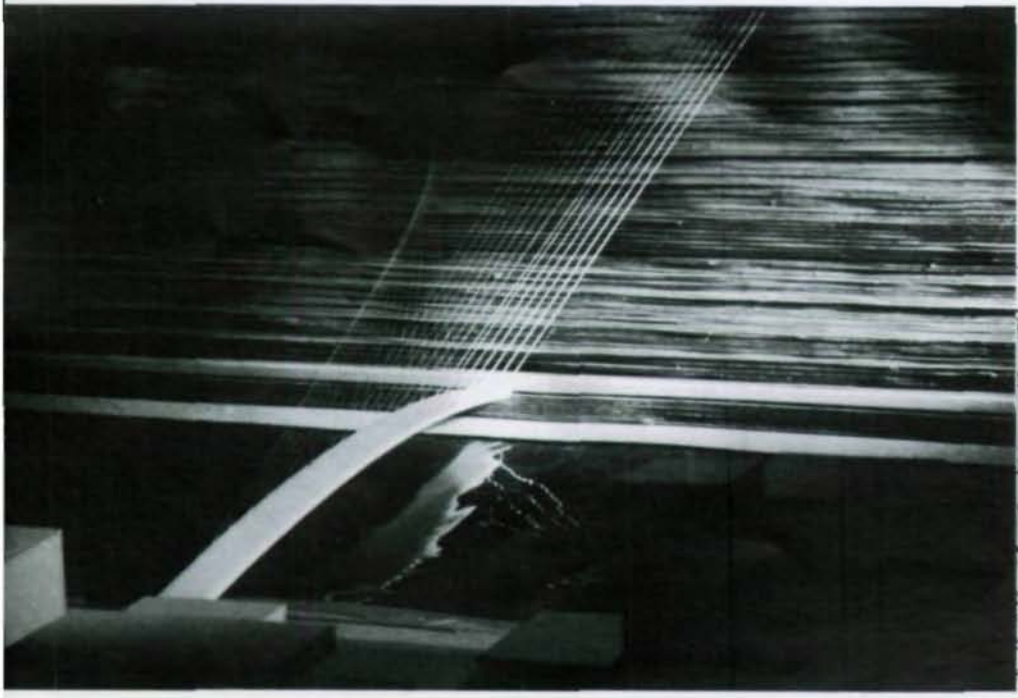
1986 "Bridges of Steel" Competition: All Bridges Lead to Pittsburgh

The flood-swollen waters of the rampaging Monongahela River destroyed Pittsburgh's historic Smithfield Street Bridge. Concern over the loss of such an historic landmark compelled a number of citizens' groups to petition the city to rebuild the bridge.

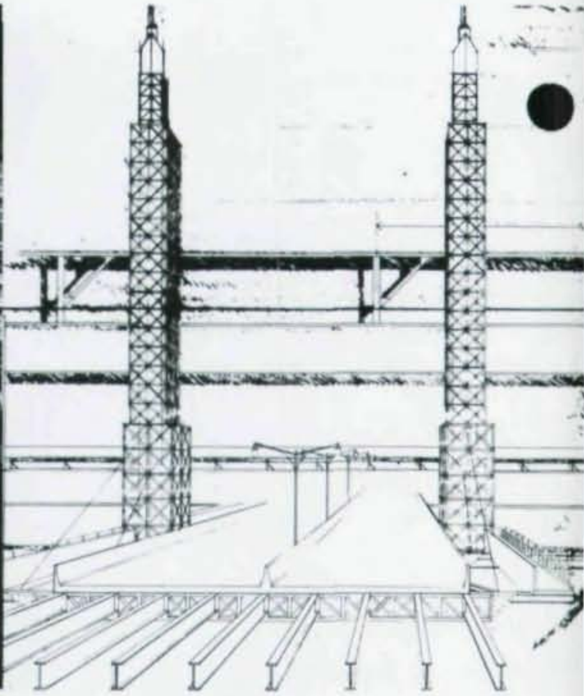
No—*it didn't happen*—only in the problem statement proposed in this year's student "Bridges of Steel" competition. The contest is conducted by the American Institute of Architecture Students and sponsored by AISC. And the "plot" just stated was given to each student as a recently hired "designer" to plan a new bridge to



Second Place—*Michael Patton and Mahesh Shrestha*



Third Place—*Charles Stetson and Thomas Hareas*



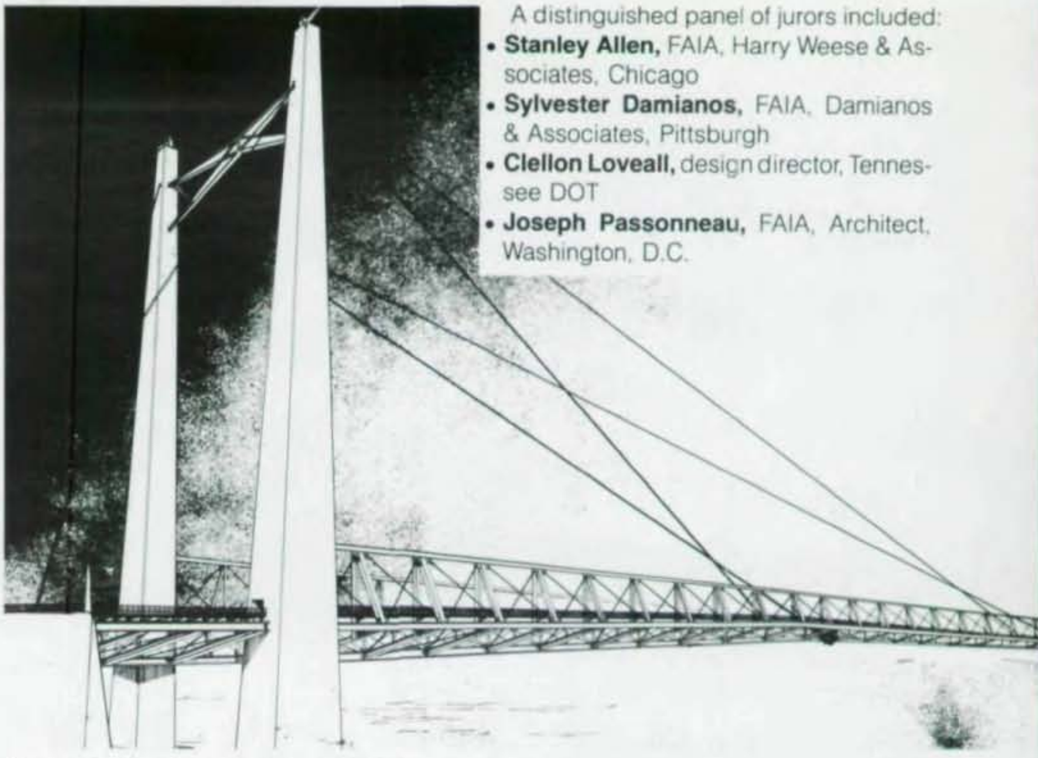
Honorable Mention—*John H. Martin*

take the mythical place of the one destroyed.

Over 100 architectural and engineering students from 70 schools submitted entries. They were to use the latest advances in theory and technology to both preserve the memory of the original bridge and design the new one aesthetically with steel. The great number of bridges that span Pittsburgh's three rivers is a source of civic pride. It was important the student designer capture that spirit as he planned a new structure for the "Bridge Capital of the World."

The entries were judged—on April 25 in Pittsburgh—on the basis of the structural and architectural use of fabricated structural steel. Their entries were to be expressive of steel construction and reflect many of the design choices made possible by using steel. Their final concept should enhance the important link between the city's Station Square and downtown, and serve as a focal point for civic discussion for years to come. First, second and third prizes were $5,000, $3,000 and $1,000 respectively, plus two $600 honorable mention awards. This competition was the first time since 1928 that bridge design was the subject of the student design contest. And the winners are. . . . .

1st Place	*Su Thanh Nguyen*, College of Architecture, University of Houston
2nd Place	*Michael C. Patton and Mahesh Shrestha,* Dept. of Architecture, Illinois Institute of Technology, Chicago
3rd Place	*Charles Stetson and Thomas Hareas,* Dept. of Architecture, Illinois Institute of Technology, Chicago
Honorable Mentions	*John H. Martin,* Graduate School of Design, Harvard University, Cambridge, Massachusetts and *Marshall Strabala,* Graduate School of Design, Harvard University



Honorable Mention—*J. Marshall Strabala*

A distinguished panel of jurors included:

- **Stanley Allen,** FAIA, Harry Weese & Associates, Chicago
- **Sylvester Damianos,** FAIA, Damianos & Associates, Pittsburgh
- **Clellon Loveall,** design director, Tennessee DOT
- **Joseph Passonneau,** FAIA, Architect, Washington, D.C.

New Marquette University College of Business Administration addition, Milwaukee, Wisc.

Marquette University:
New Addition a Lesson in Adaptability

Marquette University, a renowned educational institution, takes pride in its sprawling, urban campus with its many diverse structures. In an effort to keep its facilities in step with its program, the school maintains a conscientious construction and renovation schedule. However, to maintain the status of a private college not privy to public funds is an arduous task at best.

The original planners of the College of Business, constructed in 1949, never anticipated the growth of what was then a small Jesuit University. By the early 80's, the school had outgrown its facilities and lacked efficient circulation patterns. The present architect, working with an engineering consultant firm, took on the task of alleviating the problems. A 30,000-sq ft space for offices and classrooms was added and 50,000 sq ft of existing space remodeled entirely. The new five-story steel-framed addition abuts the six-story concrete frame of the existing structure.

Having made the decision to expand, Marquette presented the architect/engineering team with a number of challenges:

- The administrators wanted to project a professional image. The school offers a prestigious MBA program, along with numerous other programs in conjunction with Milwaukee's business community. The right image and facilities would strengthen that relationship.
- Expansive, state-of-the-art lecture halls, conference and business simulation areas require long spans of column-free space.
- Extensive involvement with the business community, in addition to regular classes, kept the school open around the clock. Construction had to be accomplished without interrupting operations.

"We created an addition that consists of a 200-person lecture hall and a 50-person executive classroom, plus the normal complement of classrooms and faculty offices," David Stroik, the architect's project manager, stated. He adds, "The remodeled part provides a computer lab and an organizational behavior lab, which accommodates realistic business situations—stock transactions through computers, audio-visual or group dynamic techniques."

A difficult design task was integration of the new with the old. By replacing some of the limestone exterior with precast, and all the existing glazing, it was possible to match simultaneously the new to the old—and to update a substantial but no longer stylish older building. The HVAC system also needed much updating. A new penthouse mechanical area was built and equipped with an energy-efficient central system.

The Choice was Steel

Stroik further adds that since the original building was concrete, it was first considered as the likely choice. However, as design required an extensive amount of irregularity in the framing, it became less viable. And a lack of repetition cut seriously into the economy of concrete. Plus, the depth of a concrete floor framing system increased as the irregularity increased.

When asked about the decision to use steel, John Komp, a structural engineer with the consulting firm commented, "Originally we felt a flat-plate design would work best. But as design progressed, column spacing became increasingly irregular, and we were forced to add concrete beams at many locations. At this point, I decided to reconsider a steel structure. The relatively small bay sizes enabled us to use a 12-in. wide-flange beam composite with the 5½-in. deck in all critical headroom areas. The result was a steel structure just as shallow as concrete, but at considerably less cost.

"Another consideration in the choice of steel," Komp adds, "was the transfer girders at the second floor. These members had limitations to provide headroom in the lecture halls below. The shallow depth requirement would have needed a very wide concrete transfer girder. In turn, this would have required staged post tensioning to properly control stresses and deflections, since it was partly exposed. The steel transfer girder alternative afforded a far narrower beam in addition to superior deflection characteristics—and was far more aesthetically appealing."

An important consideration in selecting the structural steel system was the proj-

Expansive, state-of-the-art lecture halls required long spans that steel can deliver.

Steel-framed construction occurred without disrupting building use and regardless of weather conditions.

ect's schedule and the need for continuous building operation. Stroik said, "When a campus is already cramped for space, the last thing it needs is the loss of a whole building during remodeling." Even off-hour construction was restricted because of the very active evening hours. The solution was to maintain classroom operation in the existing building during the regular school year. Then move the active programs into the new addition to permit remodeling during the summer. This timetable required every phase of construction, including the new steel-framed exit stair and addition superstructure, be completed on time—regardless of weather.

The person-to-person, person-to-computer and computer-to-computer communication was another important factor in the building design. So much so that empty conduit and buss systems criss-cross the building in anticipation of future construction needs. The structural steel framing system also permits future changes or penetrations without the fear of breaking through critical reinforcement or striking buried conduit. With steel construction, the structure and other building systems are visible, hence more adaptable.

Overall, because of the irregularities, steel framing kept costs in check—they were one-half of one percent under budget! □

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Structural Engineer
Harwood Engineering Consultants
Milwaukee, Wisconsin

Steel Fabricator/Erector
River Steel, Inc.
LaCrosse, Wisconsin

Owner
Marquette University

Exodermic deck panel is installed on Driscoll Bridge, one of 564 panels replaced.

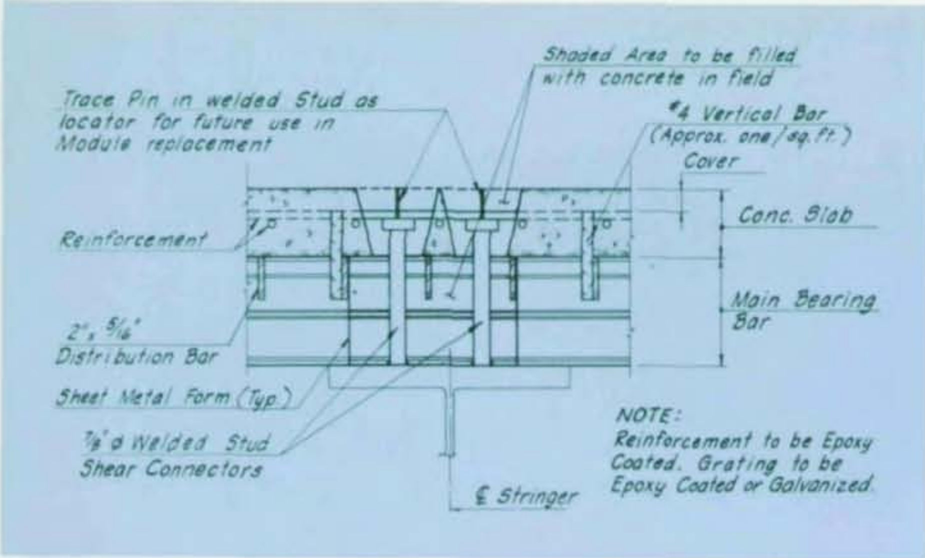
Exodermic Deck Systems: A Recent Development in Replacement Bridge Decks

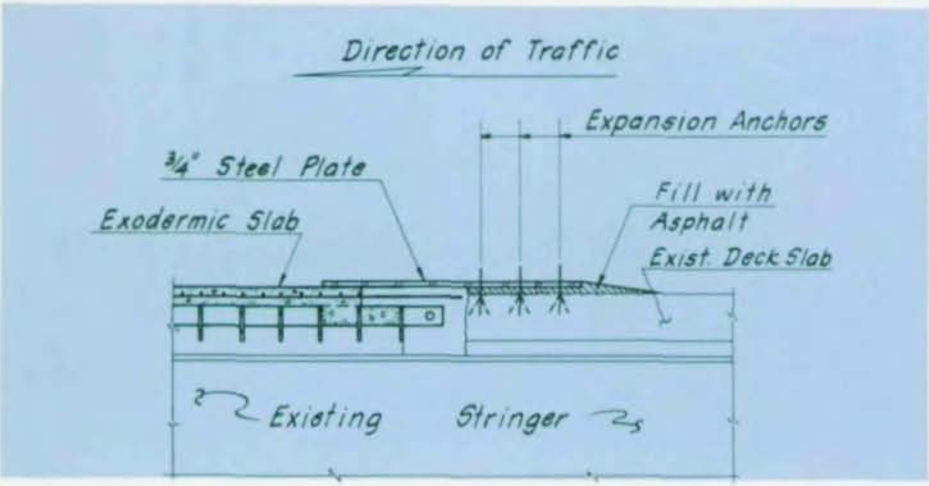
by Vincent N. Campisi

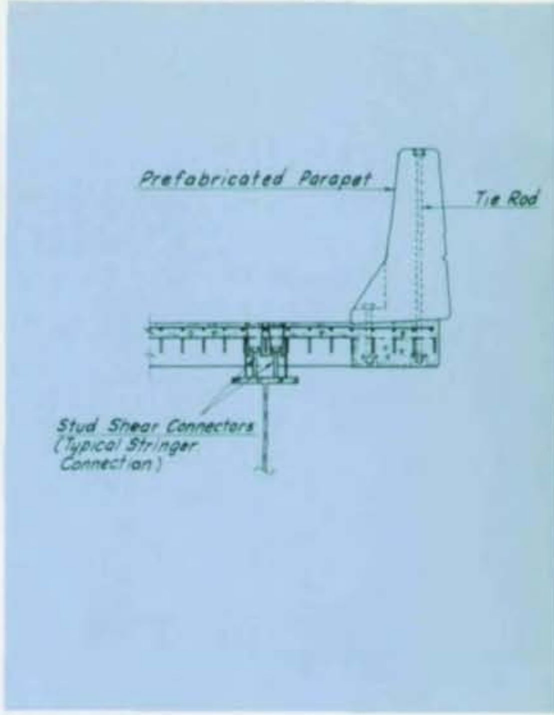
Vincent N. Campisi, P.E., is the executive director of the recently formed Exodermic Bridge Deck Institute, Inc., Westwood, New Jersey.

Recently, a major development in prefabricated deck systems has become available for bridge deck replacement projects. The three major manufacturers of steel grating have recently added to their product line a new system referred to as exodermic deck systems. This type of deck system consists of a thin, reinforced concrete overlay placed on a conventional steel grid and made composite with the grid.

The composite properties of this system are such that a bridge deck capable of carrying maximum wheel loads for spans up to 16 ft and weighing 50 psf using conventional materials is available. The system, which can be fabricated entirely in the shop, has been used on several structures







Typical connection to stringer (l., top); System for peak-hour traffic maintenance (l); Fascia detail (above).

to date, ranging in length from 80 ft to 4,400 ft. Most recently, an exodermic system was chosen for a four-span structure carrying a local road over the New York State Thruway. The structure, originally constructed in the early 50's, consists of rolled beam stringers spaced 8'-10" o.c., with an overhang of 2'-3" on each side. Fascia to fascia dimension of the structure is 31 ft out to out, well within the available length of a prefabricated module, which is about 45 ft.

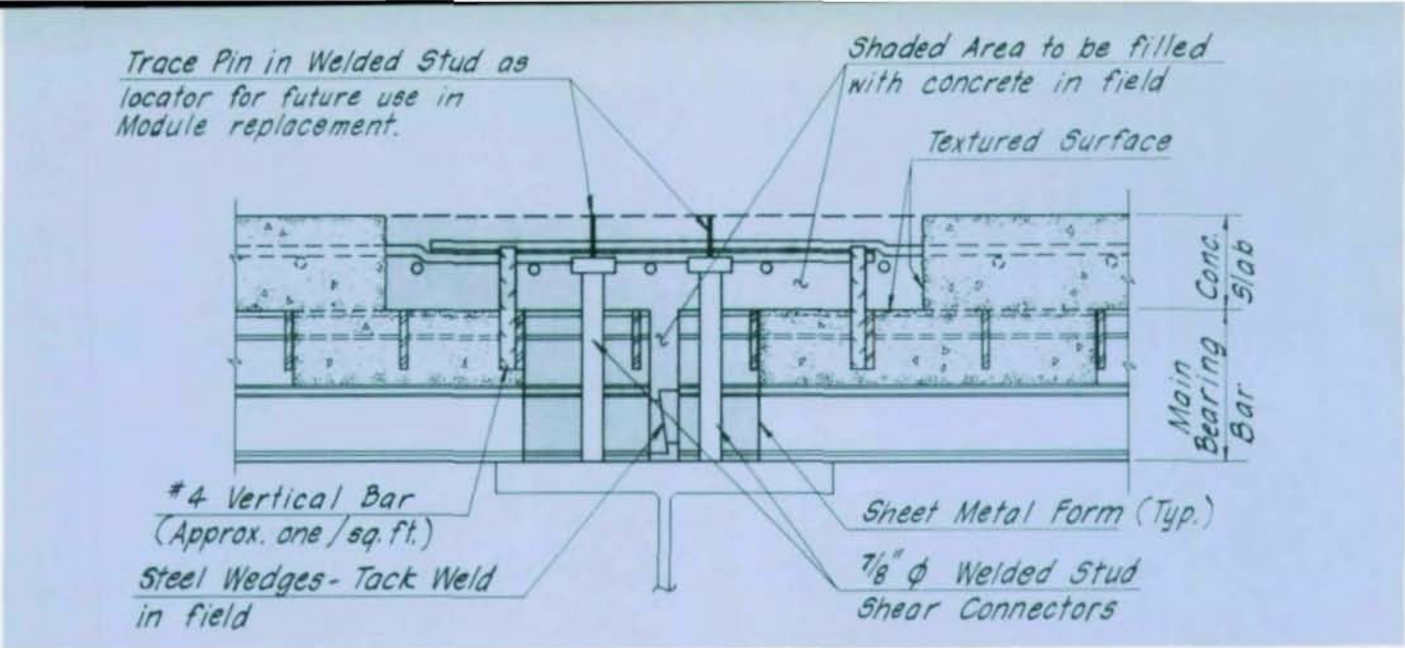
The method chosen for the deck replacement on this project was to close the structure to traffic and remove the entire existing deck. Once removed, it was estimated the exodermic deck could be placed in three days over the entire structure. In this particular case, a detour was available. In cases, however, where a detour is not feasible, the replacement deck can be placed in sections during off-peak periods, with normal traffic restored during peak periods.

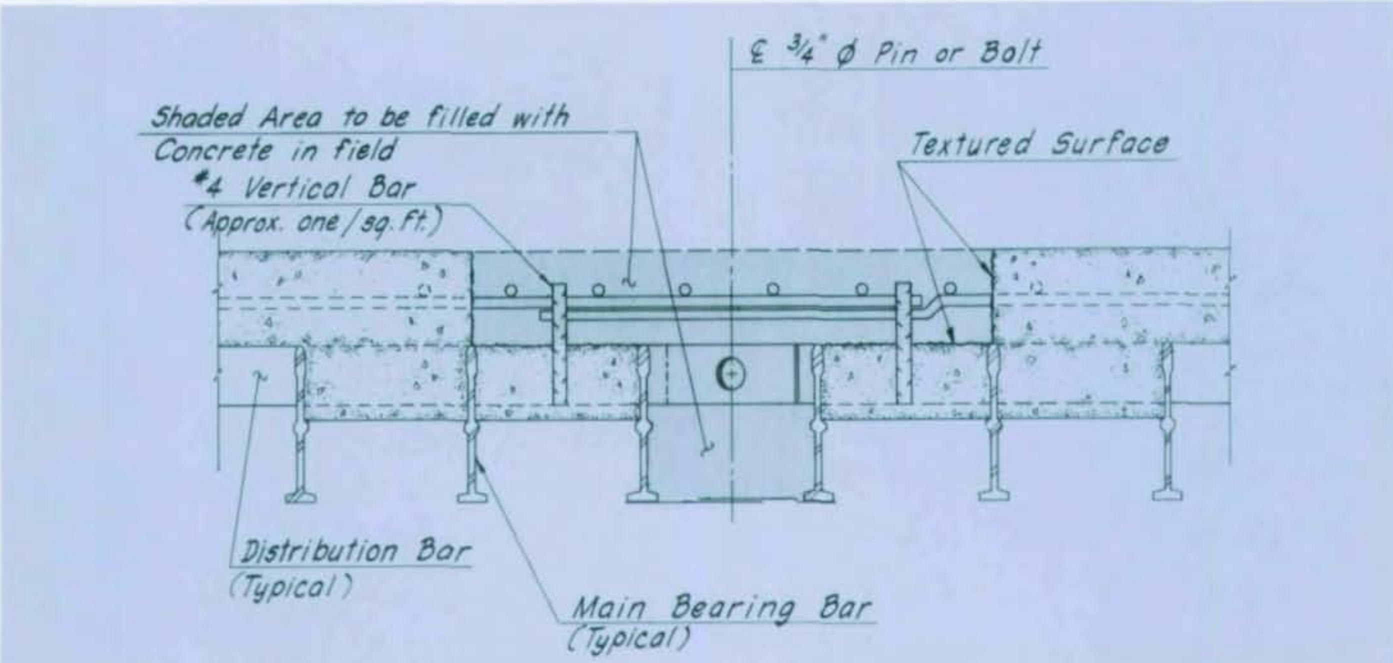
Several Advantages

Exodermic deck systems have proven advantageous for several reasons. In addition to ease and speed of erection because it does not require any staging from below the structure, the system is a high quality product fabricated in a shop condition with dependable tolerances, and a degree of quality control not easily achieved in the field. The result is a product which does not vary in strength, quality or appearance throughout the entire installation. The decks are composite within themselves. The concrete overlay, generally 3-in. thick and reinforced with a two-way layer of epoxy coated reinforcing bars, is cast onto the steel grid and attached by short vertical dowels welded to the steel grid. Horizontal shear capacity is developed through these dowels and by embedding part of the normally serrated distribution bars of the steel grid assembly.

The neutral axis of the composite assembly is designed to be located near the bottom of the concrete overlay so that the concrete acts in compression and the steel grid carries all tensile forces. Where the deck is over a support (i.e. a stringer or floorbeam) the deck is designed assuming a cracked concrete section and all tensile stresses in the concrete zone are carried by the layer of reinforcing steel. This entire system is then made composite with supporting members by attaching the deck to the member with stud shear connectors placed through openings in the precast. The area between the top of the stringer and the deck module is then filled with a grout mixture. The deck modules are set to proper line and grade using a system of leveling bolts to the corners of each panel. Adjacent panels are connected by bolts or dowels placed through slotted holes in the overlapping grid bars, and by splicing the reinforcement in the concrete overlay.

All joints are filled with the same grout mixture used for attaching the deck to the





Joint between modules, perpendicular to main bearing bars (above); parallel to bearing bars (below).

structural support system. The steel grid can be epoxy coated or galvanized, so maintenance or painting of the grid is virtually eliminated. Wearing courses of high-density concrete, latex modified concrete or bituminous wearing sufaces with membrane waterproofing systems can all be used in conjunction with this system. The system can also employ lightweight or polymer concrete where further weight reductions are advantageous.

Light Weight Important

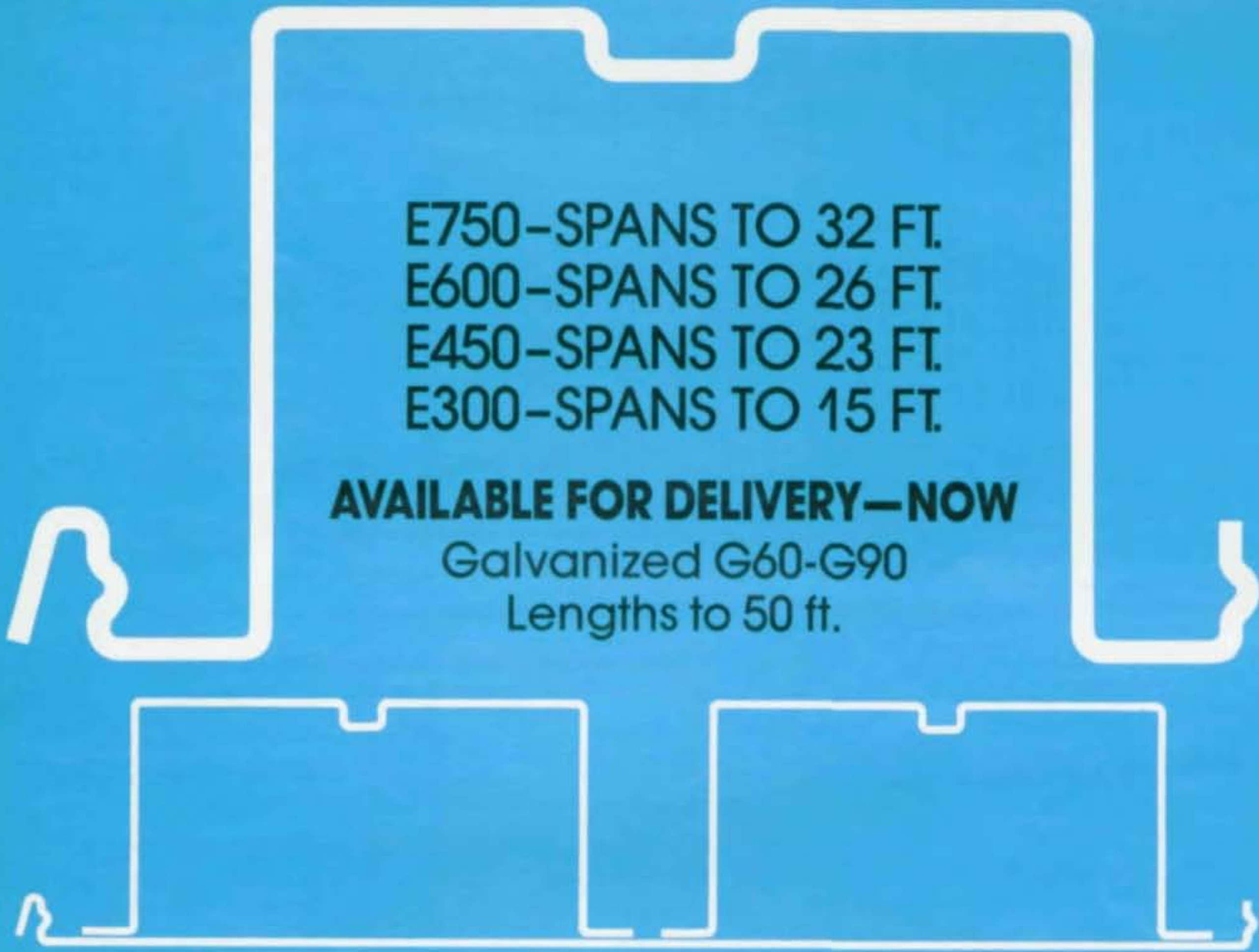
The principle of exodermic deck systems is particularly suited to bridge rehabilitation projects because of its light weight. Reduction in the deck's dead load can increase the live-load capacity of a structure; or bring a structure to its proper rating without modifying the structural support system where for one reason or another the structure does not rate properly. Overall cost savings over other methods of strengthening an existing structure can therefore be realized. In new installations, the lightweight deck can achieve savings in the weight of the stringers or girders of approximately 15% or more, depending on stringer spacings and span lengths. This represents a significant advantage to the steel industry in making steel bridges more competitive with concrete structures.

Myriad Uses

Exodermic systems can be used for other applications, including short-span bridges where the entire floor system is an exodermic deck spanning abutment to abutment; floors for industrial buildings or mechanical rooms in high-rise buildings. Other uses include piers or docks, relieving platforms, airfield runway construction, tunnel flooring, parking decks, dams, promenades and utility vault covers. Availability of numerous configurations of steel grid makes possible a wide selection of choices to insure the most economical design for each specific installation.

The Exodermic Bridge Deck Institute has recently been formed to provide technical information for design consultants, state or local governments or those in private industry who wish to consider an exodermic system for their projects. Technical literature, design manuals and specifications are being prepared and should be available in August. □

EPIC LONG-SPAN DECKS



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