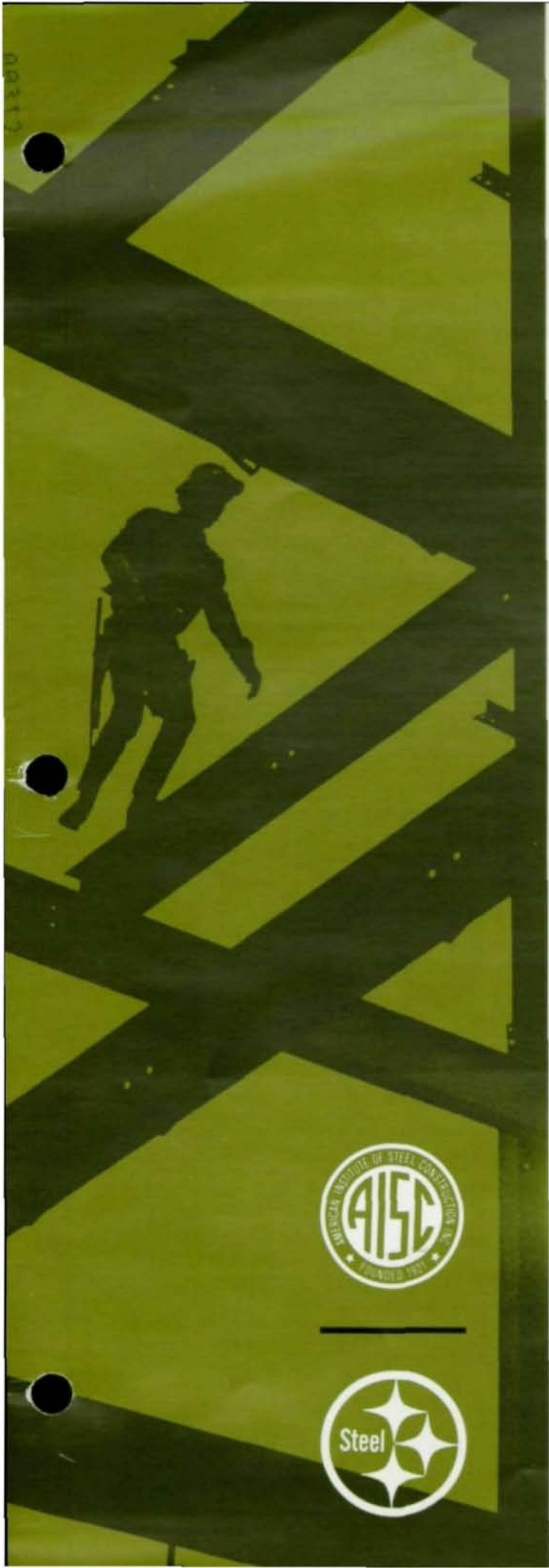


VOLUME XXIV NUMBER 1/FIRST QUARTER 1984

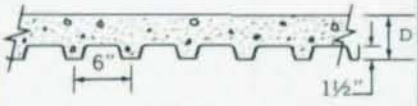
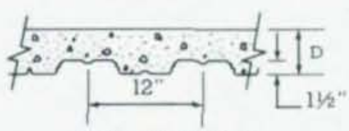
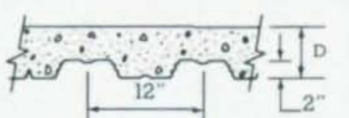
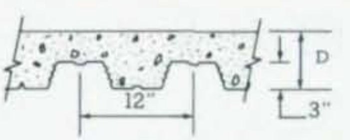
# MODERN STEEL CONSTRUCTION

An Unending Symphony in Steel
A Tale of Two Bridges
New *Engineering for Steel Construction* Released
Building with a Historical Flourish
Creating Space Where There is None
Steel Wins the Roof "Sweeps"

# 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 |
|  | 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 |
| 1½" B-LOK | 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 |
| UNITED STEEL DECK, INC. | 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 |
|  | 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 |
| 1½" LOK-FLOOR | 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 |
| UNITED STEEL DECK, INC. | 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 |
|  | 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 |
| 2" LOK-FLOOR | 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 |
| UNITED STEEL DECK, INC. | 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 |
|  | 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 |
| 3" LOK-FLOOR | 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 |
| UNITED STEEL DECK, INC. | 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.

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

# MODERN STEEL CONSTRUCTION

Published by

**American Institute of Steel Construction**
The Wrigley Building
400 North Michigan Avenue
Chicago, Illinois 60611




**REGIONAL OFFICES**

| | |
|---|---|
| NORTHEAST REGION | |
| New York, NY (Hdq.) | 212/695-4291 |
| Boston, MA | 617/329-7417 |
| Philadelphia, PA | 609/858-9354 |
| Pittsburgh, PA | 412/443-8840 |
| SOUTHERN REGION | |
| Atlanta, GA (Hdq.) | 404/458-7679 |
| Charlotte, NC | 704/541-0960 |
| Dallas, TX | 214/630-5236 |
| Houston, TX | 713/270-6363 |
| CENTRAL REGION | |
| Chicago, IL (Hdq.) | 312/670-2400 |
| Detroit, MI | 313/352-5558 |
| Minneapolis, MN | 612/888-3791 |
| St. Louis, MO | 314/721-1332 |
| WESTERN REGION | |
| Los Angeles, CA (Hdq.) | 818/444-4519 |
| Denver, CO | 303/831-4622 |
| San Francisco, CA | 415/932-0909 |
| AISC HEADQUARTERS | |
| Chicago, IL | 312/670-2400 |
| GOVERNMENTAL AFFAIRS | |
| Washington, D.C. | 202/466-5548 |


VOLUME XXIV NUMBER 1/FIRST QUARTER 1984


## CONTENTS


| | |
|---|---|
| Beth Israel: Unending Symphony in Steel | 5 |
| A Tale of Two Bridges | 10 |
| New *Engineering for Steel Construction* Released | 14 |
| 333 Wacker: Building with a Historical Flourish | 17 |
| Cedars-Sinai: Space Where There is None | 22 |
| Prestonwood Baptist: Steel Wins the Roof "Sweeps" | 26 |
| Welded Steel Duct Bridges Factory Buildings | 28 |


### 1984 PRIZE BRIDGE COMPETITION ANNOUNCED

*Entries for AISC's 52nd Prize Bridge Competition—to select the most beautiful bridges opened during the period Jan. 1, 1982 through June 30, 1984—are now being accepted. A distinguished panel of professionals has been named as the Jury of Awards:*

***Edward V. Hourigan**, Director-Structures Design and Construction, New York Dept. of Transportation, Albany, NY*
***Richard W. Karn**, President, Bissell & Karn, San Leandro, CA; President-elect of ASCE*
***Thomas R. Kuesel**, Chairman, Parsons Brinckerhoff Quade & Douglas, New York, NY*
***Charles Seim**, Principal, T. Y. Lin International, San Francisco, CA*
***Harry Weese**, Chairman, Harry Weese & Associates, Chicago, IL*

*Judging of entries will take place on September 18, and the winners will be advised shortly thereafter on the jury's decision. Winners will be featured in the December issue of* Engineering News-Record. *Award presentations to the winners will be made at AISC's prestigious Fourth Annual Awards Banquet in Chicago on December 4th.*

*All entries must be postmarked not later than September 1, 1984. Further details and entry forms may be obtained from: AISC, Awards Committee, 400 N. Michigan Avenue, Chicago, IL 60611-4185.*

### ENGINEERING FOR STEEL CONSTRUCTION NOW RELEASED!

*The all-new* Engineering for Steel Construction *is now available. A brand-new text for advanced detailers and design engineers, the book is keyed to the 8th Edition* Manual of Steel Construction. *A must for every designer's reference shelf, it contains design and detailing procedures for more complex connections and structures. 370 pages, with over 315 drawings. See the summary on page 14, and the ad on page 25 for instruction on ordering. Order today for early delivery!*

# Beth Israel Hospital: An Unending Symphony of Flexibility

**by Elliot Paul Rothman, Terry A. Louderback and Apostolos M. Antonopoulos**

The Beth Israel Hospital in Boston, Mass. is a tightly grouped complex of interconnected structures built at intervals that began in 1928. The most recent round of construction, known as "The Project," includes four major structures: Reisman, Libby, Stoneman and Northeast Buildings, a total of $44 million of construction. In addition, immediately prior to beginning design work on The Project, the team designed the new $12-million Dana Biomedical Research Facility, also at the Beth Israel Hospital. Each new structure required innovative solutions to varying sets of problems.

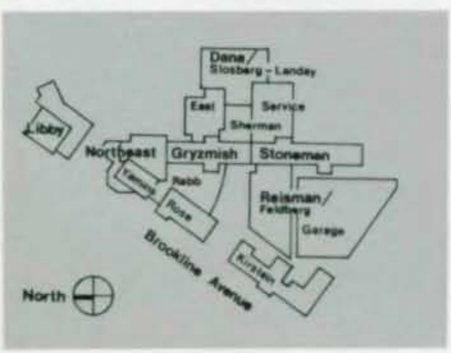
### Lightweight Construction Gains 35,000 Sq Ft Not Otherwise Possible

For the Dana Biomedical Research Building, the problem was to maximize the total floor area above the existing four-story Slosberg-Landay Building which was originally designed for flexibility and future vertical expansion of four new floors. Member sizes were based on future construction with the same configuration as the original design, i.e. 4½-in. solid one-way slabs on non-composite structural steel beams. To maximize the number of additional floors, yet maintain the quality of the original construction, several alternative framing schemes were investigated.


*Elliot Paul Rothman*, A.I.A., is a director of the architectural firm of Martha L. Rothman-Elliot Paul Rothman Inc., Boston, Massachusetts.

*Terry A. Louderback*, P.E., is a principal in the structural engineering firm of Souza and True Inc., Watertown, Massachusetts.

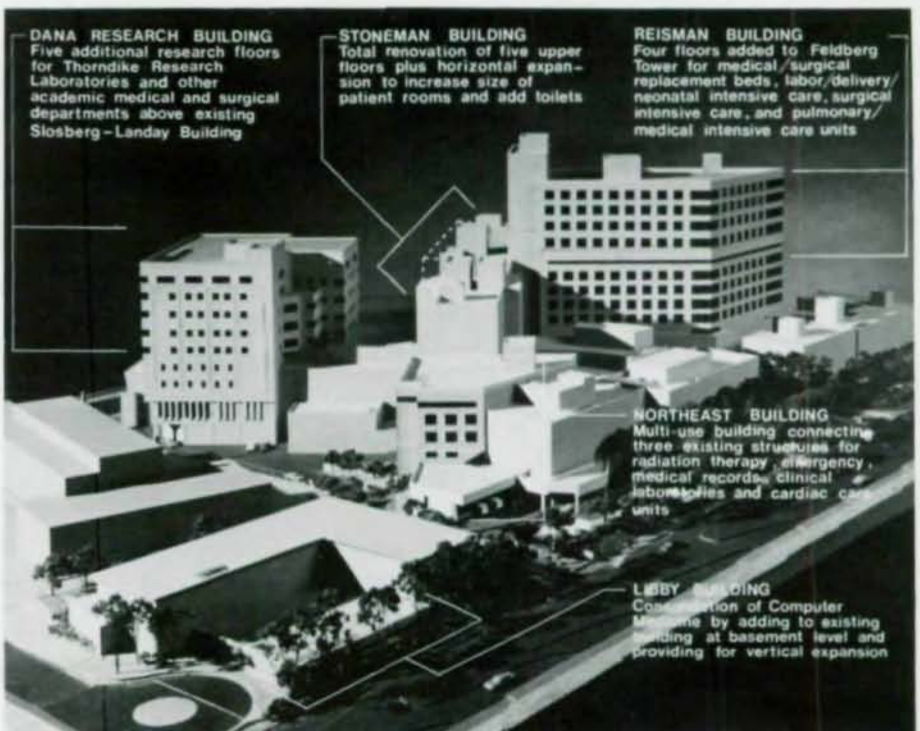
*Apostolos M. Antonopoulos* is an engineer in the structural engineering firm of Souza and True Inc., Watertown, Massachusetts.


The scheme chosen utilizes 3¼-in. lightweight concrete topping on 2-in. composite steel deck supported by composite structural steel beams and girders. The scheme offered lightweight, shallow structural depth and a relatively high degree of rigidity for the moment-resisting frames. By introducing light gauge steel studs as backup for the exterior brick as well as at critical interior partitions, the team was able to provide the owner with one more floor (about 17,000 sq ft) than was possible using the original type of construction—and without compromising the sound floor construction the owner and team demanded. As a result, five new floors were provided instead of four.

One of the interesting structural features of this building was that the fabricator elected to fabricate columns with cantilevered girders as "trees" so the girder section could be continuous through the column while the columns were spliced at mid-height. This resulted in a shop-welded moment connection (the structure resists lateral forces by moment-resisting frames) between the ends of each column and the flanges of each cantilevered girder, with appropriate stiffener plates.

The Reisman Building is similar to the Dana Research Building in its vertical expansion upon the original steel frame designed for future loads. Here, however, the design problem was somewhat different. The original lateral force resisting system (steel moment-resisting frames) was based on a maximum ultimate building height of approximately 110 ft above the roof of the original design. Introducing lightweight materials, such as composite steel girders and beams supporting composite steel decks plus lightweight concrete floor topping and light gauge steel studs, the floor-to-floor height was kept to absolute minimum (12 ft on upper floors). Nine additional stories were built upon the existing structure, instead of the eight possible if new construction was the same as the original design. That is a net increase




Boston's Beth Israel Hospital. Model photo and legends show multiplicity of remodeling. Campus plan at left. Photo by Peter Vanderwarker.

of 18,000 sq ft of floor area. These reduced floor-to-floor heights required very close coordination among team members. Under the present building program, the hospital is constructing four new floors, plus numerous rooftop penthouses, the roofs of which can serve as future floors.

The Libby Building Computer Medicine Facility, partially below grade, is a one-story reinforced concrete structure attached to the Libby Building and connected to the hospital complex by a tunnel. The structure was designed for future vertical expansion of three additional floors.

### Innovative Upgrading of Existing Patient Rooms to Standards

The Stoneman Building will be expanded laterally to provide more spacious patient rooms consistent with rooms in the Reisman Building. The structural solution to enlarging the patient rooms in the Stoneman Building evolved from the criteria that the east rooms be expanded by 8 ft, but only from the fourth floor up. No new structure could be permitted below. By using trussed supports at each existing column, and interconnecting these supports from floor to floor, the new loads are transferred through bearing at the ends of the compression diagonals into the existing columns. Cantilever moments, due to new construction, are transferred into the existing structure at each floor (and roof) by horizontal compression and tension forces. The horizontal components of the compression force on the diagonal struts tend to negate the tension force in the horizontal members at all levels, except the roof and lowest floor, where special connections are provided to transfer unbalanced horizontal forces. Construction is light weight to minimize the increased load on the existing structure. Each column and footing in the existing structure affected by the new construction was checked and found to have adequate reserve strength to safely support the new loads.

### Steel Solves Complex Building Requirements

Perhaps the most challenging structural problem was the Northeast Building. This new structure is contiguous to three existing buildings—Rabb, Gryzmish and Yamins—each of a different type and era of construction. Among the important considerations in the selection of a structural system were: 1) relatively shallow construction was required to provide ample headroom while matching the floor-to-floor heights of the existing buildings; 2) certain portions of the existing buildings, which were eventually to be demolished and replaced with new construction, were to remain in operation while construction of the new building was underway; 3) the multi-use nature of the new building required different live loads at different floors, but for coordination reasons the depth of structure was to remain constant. This characteristic was critical in coordinating the work of the mechanical, plumbing and electrical trades.

Steel framing of Northeast Building met most challenging structural problems in complex remodeling.

Given the above criteria, several framing schemes, both in concrete and in steel, were designed and priced. The preliminary conclusion was that for the superstructure, the most economical design appeared to be a unidepth, one-way, reinforced concrete, ribbed slab, supported by wide reinforced concrete girders. The relative costs of the different systems were based on costs of materials at the time of the preliminary designs.

Events that took place as the design of the Northeast Building evolved clearly indicate the flexibility and cooperation a design team must display to achieve a successful project. Following the preliminary drawing stage, the design process continued to assess more factors upon which the choice between structural steel and concrete would depend. These factors were grouped into three broad categories: super-structure, phasing and foundations.

**Ease of Super-Structure Construction Next to Existing Buildings**

One of the earliest factors which led to a re-evaluation of the original concrete design was that construction of a portion of the Northeast Building would extend over the existing Rabb Building. The Rabb Building had been designed for future expansion, but not to the degree required for the Northeast Building. The existing columns of the Rabb Building were not originally designed to support a concrete structure. The possibility of constructing new Northeast Building columns down through the Rabb Building was investigated, but rejected for several reasons. Amongst the reasons was that foundations for new columns would have to be built 10 to 15 ft below the lowest floor level at Rabb and adjacent to large air handling equipment which had to remain in operation.

The simplest and most practical solution seemed to be to build on the existing steel columns, using their additional capacity. The only way to accomplish this was to use lightweight structural steel. Therefore, it was decided to change half of the first bay adjacent to the Rabb and Gryzmish buildings from concrete to structural steel.

**Phasing**

The need to maintain access to the Berenson Emergency Unit had a tremendous impact on construction of the Northeast Building. A corridor had to be maintained from the temporary entrance at the Yamins Building to the emergency unit in the Gryzmish Building. In two areas, the first floor of the Northeast Building overlapped the emergency corridor. With the concrete scheme, these areas would have been omitted and placed later inside a complete building. Two columns adjacent to the Gryzmish Building, one on either side of the emergency corridor, were changed from concrete to steel so they could be dropped into place. The space available for construction did not allow enough room even for formwork.

Design criteria for Northeast Building

DESIGN CRITERIA

Notes

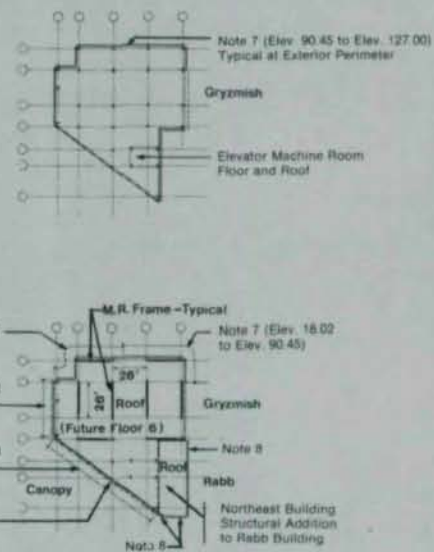
1. Design loads shown in pounds per square foot.
2. For allowable future column loads see column schedule.
3. Dead load allowance for future floor construction based on presently shown composition slab system.
4. Dead load allowance for future roof construction based on steel deck/beam system shown for roof of addition to RABB BUILDING.
5. Superimposed dead load is the total weight allowance for all permanent construction attached to or hung from any floor or roof, including ceiling, mechanical, plumbing, electrical, or other, exclusive of partition live and dead load of structure.
6. Future expansion over structural addition to RABB has *not* been provided for.
7. Total weight of exterior curtain wall system, including exterior cladding and back-up shall not exceed 50 p.s.f.
8. Total weight of exterior curtain wall system, including exterior cladding and back-up shall not exceed 15 p.s.f.

**Future Construction**

| | DESIGN LOADS | | |
|---|---|---|---|
| Level | Live (Roof) | Partition | Superimposed Dead Load |
| Elevator Roof | (30) | – | 35 |
| Elevator Mach. Rm. | Elevator Design Loads (see elevator drawings) | | |
| Roof Level 9 | (30min) | – | 35 |
| Floor 8 | 60 | 20 | 20 |
| Floor 7 | 60 | 20 | 20 |

**Present Construction**

| | DESIGN LOADS | | |
|---|---|---|---|
| Level | Live (Roof) | Partition | Superimposed Dead Load |
| Roof (Future Flr.6) | 60 | 20 | 20 |
| Roof | (30min) | – | 35 |
| Mechanical Floor 5 | 150 | – | 20 |
| Floor 4 | 60 | 20 | 20 |
| Floor 3 | 60 | 20 | 20 |
| Med. Rec.'s Floor 2 | 150 | – | 20 |
| E.R./Admin. Floor 1 | 100 | – | 20 |
| Floor B | 60 | 20 | 20 |



Northeast Building

The construction manager, after studying the conflicts between new and existing construction, and the phasing difficulties, suggested that, for ease of construction, the rest of the bay adjacent to the Gryzmish Building be converted to structural steel. The columns, girders and beams could be erected under, over and around the emergency corridor and the slab and deck placed later when the corridor was no longer needed.

A major element of the Northeast Building and the Emergency Unit is the entrance canopy, which provides for covered ambulance parking and mechanical areas. As the canopy design developed, the consensus of structure, given the long spans required and the limestone facade, was structural steel.

**Effect on Foundations**

The last factor which probably had the greatest impact on the choice of structure was the building foundations. With a cast-in-place concrete structure, very large spread footings, or possibly even a mat—or a combination of the two—would have been required for foundations. Construction of the footings would have required general site excavation about 15 ft below the lowest level of the Northeast Building. Site excavation would have required general dewatering of the site and the driving of sheet piling to maintain an access road from the street along the Northeast Building to the rear of the Beth Israel site. To reduce the required excavation and eliminate the sheeting and dewatering, engineers considered caissons. With the concrete building, a low allowable bearing value of 3.5 tons per sq ft, and the depth of clay available for construction of the bells, caissons would not work.

***Steel Accommodates Multi-use Buildings***

All factors indicated a re-evaluation from concrete was required. A more complete design was completed, which was specific for two of the typical floors. As mentioned earlier, different live loads were required at various floors. The second and fifth floors required live loads of 150 psf. The remaining floors needed only 60 psf. The lightly loaded floors were designed with composite beams generally at third points of the bay, W14 with 2-in. deck and 4¼-in. lightweight concrete. On heavily loaded floors, the same slab and deck were used as well as the same beam design, but the beam spacing was decreased from third points to quarter points of the bay. The same size W14 was used throughout the building. Resultant floor depth was 20 in., the same as for the concrete scheme. The key to the success of the structural steel scheme still was its compatibility with both the architecture and the mechanical work.

**Creative Detailing in Structural Steel**

Architectural requirements dictated that moment-resisting frames should be used for lateral bracing. The number of frames required was determined, and the frames were located so there was a path for mechanical ducts to travel under the 20-in. deep structure. At points where ducts crossed under heavy girders of the moment frame, close scrutiny of mechanical design and the architecture permitted 21-in. deep girders. By raising the moment frame girders 3 in. above the soffit of the deck, 24-in. deep girders could be used, greatly reducing the required weight of steel. The deeper raised girders, besides having smaller flanges which made the moment connections more economical, also made the frames stiffer and reduced fabrication costs of the beams. Because the top of the girder was 3 in. above the top of the beam, coping at the top beam flanges was not required (Fig. 1).

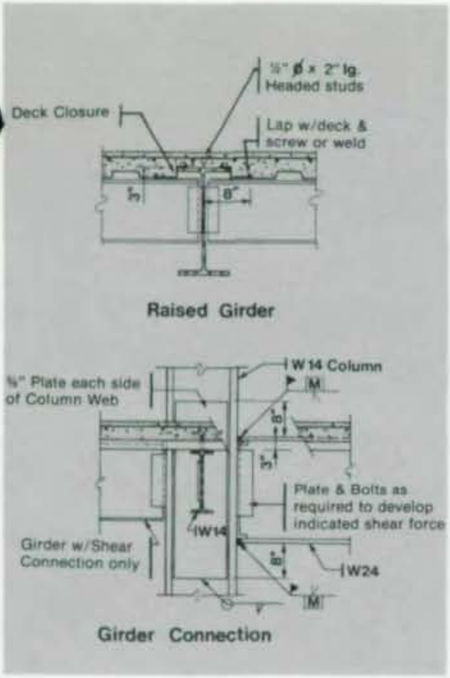



Figure 1. Northeast Building raised girder and girder connection details

In the Northeast, as well as in the Reisman Building, brick relief at each floor is continuous and was achieved without the need for diagonal "kickers" which often interfere with services in the ceiling space. Vertical channels supporting the horizontal angles are supported from perimeter beams and girders by cantilevered wide-flange stubs which are, in turn, backed up by beams framing perpendicular to the perimeter beams or girders (Fig. 2). For perimeter beams, which span parallel with floor beams, supplementary backup beams were added to align with each relief angle support.

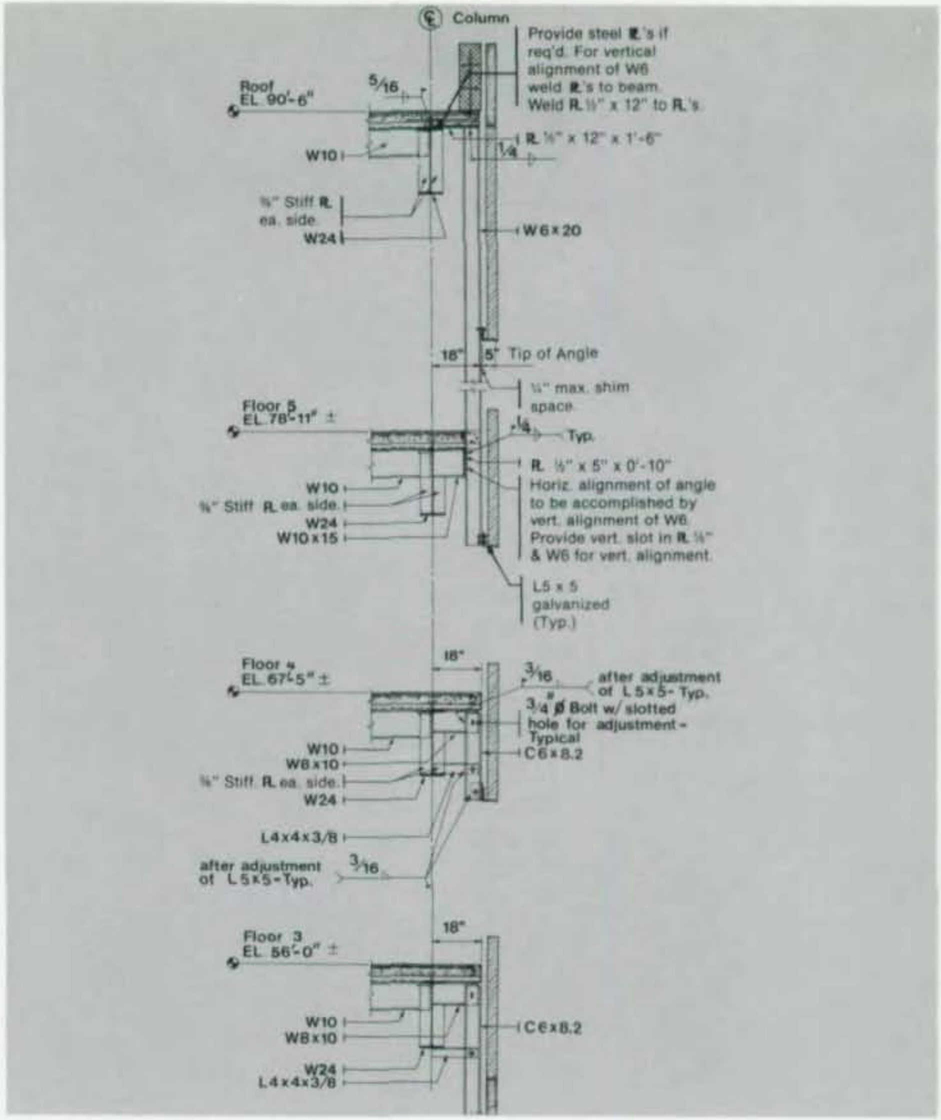



Figure 2. Exterior wall section shows relieving angle suspended from cantilevered wide-flange stubs.

**Architect**
Martha L. Rothman-Elliot Paul Rothman Inc.
Boston, Massachusetts

**Structural Engineer**
Souza and True Inc.
Watertown, Massachusetts

**Construction Manager**
Jackson Construction Company
Dedham, Massachusetts

**Geotechnical Consultant**
Geotechnical Engineers Inc.
Winchester, Massachusetts

**Steel Fabricators**
Montague-Betts Company Inc. (The Project)
Lynchburg, Virginia

Haarmaan Steel Corporation (Dana Building)
Chicopee, Massachusetts

**Steel Erector**
Daniel Marr & Son Company
Boston, Massachusetts

**Owner**
Beth Israel Hospital
Boston, Massachusetts

### Advantages of Steel Construction

By changing the structure from concrete to steel, caissons became a viable foundation scheme. Consequently, general site excavation was reduced and the requirements for site dewatering and sheet piling were eliminated. Deep cantilevered grade beams required to support new columns immediately adjacent to the existing structures were reduced significantly in depth.

After the design team identified a structural system compatible with architectural and mechanical systems, the construction manager could develop a total structural estimate which accounted for the cost of structure, both superstructure and foundations, and time of completion. The CM's analysis indicated clearly the best material was structural steel. The change from concrete to steel became even more favorable when steel prices dropped during the time between preliminaries and re-evaluation.

When the structural portions of the Northeast Building were completed, the CM had saved nearly three months of construction time by using steel rather than concrete. And, in addition to the time saved, preliminary estimates indicated that from $500,000 to $750,000 was saved in the cost of the total structure.

The Northeast Building, and the sequence of events which led to its completion, affords a good example of the flexibility and the creativity possible in construction with structural steel. All of the new buildings at Beth Israel Hospital, each with its unique set of structural design problems, offered a challenging assignment to the design team. Structural steel has been used effectively to provide the owner with well-engineered, economical and functional spaces. □

## Bridge Rehab with Speed and Low Cost . . .

# A Tale of Two Bridges

### 1. Railroad Bridge in Missouri . . .

On May 3, 1982, a middle span of the Norfolk & Western RR bridge at Hannibal, Mo. was struck by out-of-control barges. One span went into the Mississippi River and the end post of a second span was damaged. The steel structure, in service since 1887, is a part of the Norfolk & Western line linking Kansas City and the Chicago/Detroit area. Rail traffic was significantly impacted, and it was necessary to divert traffic south through St. Louis, with added expense and extended schedules.

Norfolk & Western faced the task of repairing or replacing the damaged structure in the shortest time and at a reasonable cost. They explored these possibilities:

1. Repair the damaged section of the existing bridge.
2. Replace only the damaged part of the bridge while using the undamaged structure.
3. Construct an entirely new bridge.
4. Search for a completed structure that was no longer in use, or in limited use. This structure would be purchased and altered to meet the railroad's needs.

Alternate one was eliminated due to the extensive damage and the associated costs of salvaging material. Alternate three was not feasible for schedule reasons. Alternate four was explored, but such a structure could not be found.

Replacing the damaged span with a steel structure proved the only feasible alternative within the time constraints.

Immediately after the accident, Norfolk & Western interviewed several engineering firms for designing a new truss span.

Runaway barges badly damaged Norfolk & Western RR bridge at Hannibal, MO.

Howard Needles Tammen & Bergendoff was selected because they had recently designed a similar truss bridge that could easily adapt to the railroad's needs.

While the engineer was being selected, railroad personnel concurrently negotiated with several steel fabricators. Fabricators were asked to quote a price and construction schedule based on a very open scope of work and without benefit of bid drawings. Requirements were described verbally by the owner and engineer, and the fabricators had to depend on past experience to develop a proposal.

Norfolk & Western awarded the construction contract just three days after the accident based on both price and schedule. That same evening, the fabricator's representatives met with the engineer to modify details of the similar HNTB design to comply with the railroad's. The previous HNTB design was based on a 250-ft span, whereas the damaged bridge was only 246 ft-3 in.

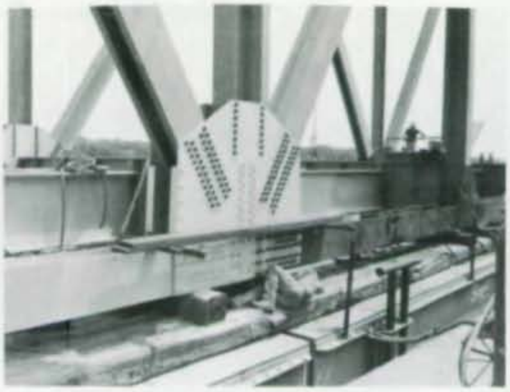
Shop detail drawings and calculations were completed by in-house Bristol Engineering and submitted on schedule to the engineer within three weeks. The fabrication schedule to commence delivery in early July could only be accomplished by fast-tracking the production process. The erection sequence, established early in the contract, allowed production to assign a priority number to each shipping piece. Both trusses of the structure were preassembled and knocked down in the shop to insure field fit-up.

Fabricated material started shipping to the site on July 16. Crews were already mobilized and immediately started assembly of the bridge structure on three barges. When the assembly was completed, the barges were floated near the removed span, and the bridge was hoisted as a complete unit. The erector used a barge-mounted Manitowoc 4000 to assemble the truss structure on barges. A derrick barge lifted the 465-ton structure into place on August 2.

A special problem that had to be solved was the need to design different bridge bearings for the new structure. The new truss span was slightly deeper than the existing one, thus the difference in depth had to be taken up by the bearings so the tracks would align. Also, additional time was gained in the field by the decision to lift the assembled unit with a crane rather than jack the unit into place. This required less specialized equipment and consequently less time for field mobilization.

And the railroad was completely operational exactly three months after the accident! □

Truss span was field-erected on barges one mile north of bridge. Total erection time was eight working days.



Field bolting of panel point is completed. Over 32,000 7/8-in. A325 high-strength bolts were used on bridge.

A BN-designed bridge was modified by removing 6 in. between each panel point.

New truss is lifted to clear piers (above), swung into place and lowered onto bearings (l.). Total lift time: 45 minutes. Within six hours first train passed over new span.

**Structural Engineer**
Howard Needles Tammen & Bergendoff
Kansas City, Missouri

**Steel Fabricator**
Bristol Steel Corporation
St. Louis, Missouri

**Steel Erector**
Bristol Steel Corporation
Bristol, Virginia

**Owner**
Norfolk & Western Railroad
Roanoke, Virginia

## 2. Century-Old Truss Bridge in Pennsylvania . . .

A century-old steel truss bridge in Coudersport, Pa. has been rehabilitated to six times its previously permitted load-carrying capacity. And the job was done at a cost of only one third that of a new bridge replacement to meet all modern standards.

The Coudersport span is the first application of the unique truss bridge reinforcing system introduced last year by two civil engineering professors at Bucknell University, Lewisburg, Pa. under a research grant from the Structural Steel and Steel Plate Producers of AISI.

Already, a second application is the rehabilitation of the Roaring Creek Bridge, built in 1890, near Elysburg, Pa. In addition, consideration of the plan is underway by counties, consulting engineering firms and state highway departments from Maryland to California. With thousands of old truss bridges, erected in the late 1800's and early 1900's, now in seriously deteriorated condition, the plan's low cost appeals to many localities which do not have funds for new bridge construction.

Build in 1883, the single-lane Seventh Street Bridge was one of two that provide local access across the headwaters of the Allegheny River in Coudersport. The second bridge, too deteriorated for repair, has been closed. It was judged possible to save the aging Seventh Street Bridge, despite buckled floor beams (restrained by cables) and a load limit lowered to only three tons. This low capacity meant that no fire trucks, garbage vehicles, ambulances or school buses could enter that part of the city.

Since there was no available funding for a new bridge, the Coudersport council chose another remedy—a truss bridge reinforcement plan created by Dr. Robert J. Brungraber, presidential professor of civil engineering at Bucknell, Dr. Jai B. Kim, chairman of its Department of Civil Engineering, and an undergraduate student assistant, John Yadlosky. The underlying concept of the technique is that the combination of a reinforcing arch with an existing truss system can carry a significant extra load if it is well-supported laterally.

Total cost of the complete rehabilitation—funded by fuel tax money—was \$62,400. A new bridge at the site, a two-and-a-half-lane facility to meet today's required specifications, would have cost nearly \$200,000. Besides the cost advantage of this plan, a primary benefit is its erection speed and a minimal interruption of traffic. The steel builder/erector met a 60-day time limit to both fabricate and install the bridge components. Actual erection time, by a crew of four, was only three weeks. The *maximum* traffic interruption was seven minutes!

New life for rehabilitated 100-year old truss bridge at Coudersport, Pa. (below). Bridge can carry six times previous load limit. Closeup of new steel reinforcing arch (l.) that made it possible.

Reinforcing steels were high-strength A572 Gr.50 for arch sections and floor beams and A36 steel for hangers and stringers. Steels were generally fabricated as structural channels, I-beams, wide-flange beams and 1-in. square bars.

The bridge, which most recently had a three-ton load limit, is now legally posted for a 20-ton capacity. Developers of the plan anticipate that, with reasonable maintenance and traffic kept at or under the new load limit, the bridge could well have a new service life expectancy of another century.

Rehabilitation began with replumbing and straightening of the bridge with chain hoists. Improved connections were provided for the end portal bracing that pulled the span back into shape. Installation then entailed erection of a new structural arch on each side of the bridge and reinforcement of a few critical members with prefabricated parts. Then, new hangers, stringers and floor beams, as well as replacements for the original buckled floor beams, were installed. Finally, a slight upward camber of the roadway was introduced for appearance.

The principle of the system is not merely reinforcement of the old bridge, but virtual replacement with a new arch bridge. In the usual repair/rehabilitation schemes, the weakest links are strengthened to restore the carrying capacity of the original structure, often less than warranted by current traffic needs. In this arch-dependent rehabilitation, the carrying capacity of the entire structure was upgraded, thus allowing live loads to be increased.

The U.S. Department of Transportation classifies 45% of all bridges as deficient or obsolete. This plan is said to be practical for all through-truss bridges in this condition.

The system does not make new bridges out of old ones. The Seventh Street Bridge remains the same in its vertical clearance, deck width, waterway opening and approach roadway alignment.

But the big improvement is in structural adequacy for modern loads. Structural steel combined with engineering innovation made it all possible! □

**Structural Engineers**
Robert Brungraber and Jai B. Kim
Bucknell University
Lewisburg, Pennsylvania

**Steel Fabricator/Erector**
Williamsport Fabricators, Inc.
Williamsport, Pennsylvania

**Owner**
Borough of Coudersport, Pennsylvania

# The most cost efficient floor system you can use.

## "We cast this Hambro® composite floor today. We'll strip it tomorrow."

Robert Satter, President
The Satter Companies, Inc., W. Palm Beach, FL.

Greenspoint I, Houston, TX
Arch: S.I. Morris
Engr: Walter P. Moore & Co.
Cont: McGregor Construction Co.
Dvlpr: Friendswood Development Co. (Exxon)

GTE/Hines, Westboro, MA
Arch: Drummey, Rosane, Anderson
Engr: Anderson-Nichols & Co.
Cont: Vappi & Co., Inc.

"The Canam Hambro® D-500 composite floor system saved us time, material and money when we used it to build our corporate headquarters, Congress Park IV," says Robert Satter. "We cast a floor one day and stripped it just 24 hours later. By then, it was ready for the sub trades to use as a work platform. The plywood and roll bars are reusable. And the job required no bridging, or on-site welding."

Hambro's unique U.L. fire-rated floor uses high strength steel. It is twice as rigid as conventional structures. Speedy erection reduces financing costs. Ideal for high or low rise commercial and residential buildings. And its sound ratings are superior. Another concrete reason to specify Hambro is that we can guarantee fabrication within three weeks of approval of final drawings.

Call or write today for D-500 Brochure. Canam Hambro, 140 Gould St., Needham Heights, MA 02194. (617) 444-5504. Telex: 95-1509.

**hambro**

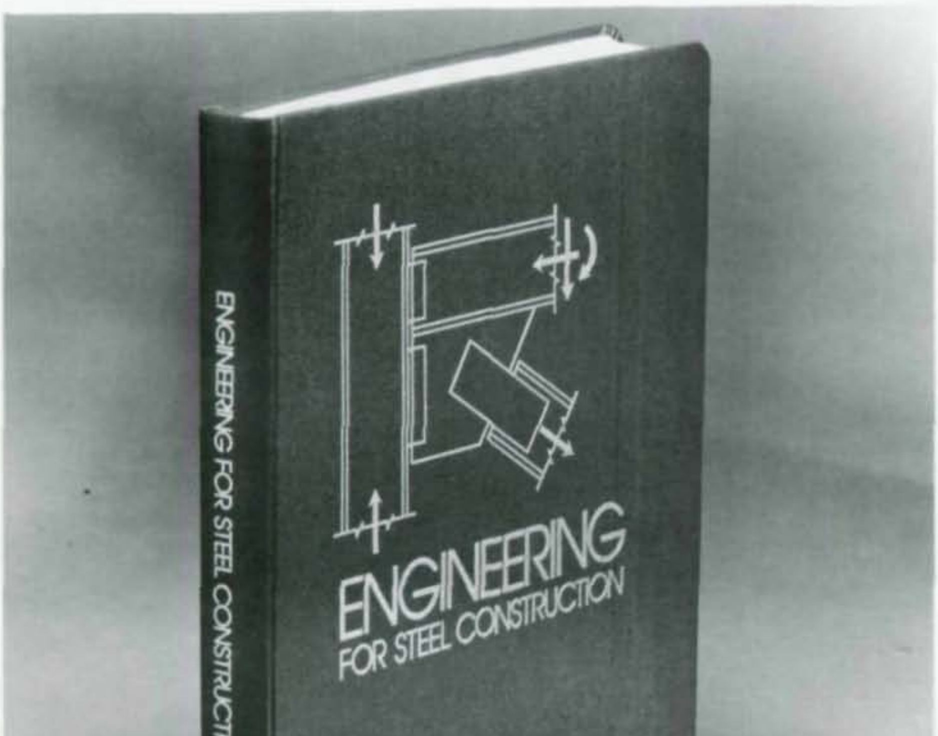
**We put a ceiling on the cost of floor construction.**

*Engineering for Steel Construction,* just published by AISC, delineates the most recent advances in connections and detailing for structural engineers. Here's a detailed summary. . . .

# *Engineering for Steel Construction*

*Reprinted courtesy* Civil Engineering Magazine, *January 1984.*

**by Robert O. Disque**




*Robert O. Disque* is Assistant Director of Engineering, American Institute of Steel Construction, Chicago, Illinois

Structural engineers are increasingly aware of the importance of details and connections in steel structures. These are not only the key to economy but also may be the source of trouble.

For this reason, AISC has just released a new 370-pg. textbook—*Engineering for Steel Construction*—for the advanced detailer and design engineer. The new book is a companion to *Detailing for Steel Construction*, which was written for the beginning detailer who might have a high school education. Since both books are self-contained, some material may be common to both. Together, the books, which replace the 2nd Edition of *Structural Steel Detailing*, are geared to the 8th Edition *Manual of Steel Construction.*

**Chapter 1:** "Structural Engineering" presents the theory and methods of application used in the book to model design procedures for details and connections. The instantaneous center method for analyzing certain types of eccentric connections is described, and references cited for simple solutions not available when the Manual was published.

Brackets are analyzed and detailed in two ways. A very conservative approach assumes the neutral axis of the bracket coincides with the center of gravity of the bolt group. A second approach determines the center of rotation by an iterative process where the static moment of the tension area equals that of the compression area. Both procedures, which result in a more realistic design, are included in the book.

Prying action is fully explained and its various applications described. The procedure in the text is based on that in the 8th Edition Manual. Prying action, essentially, is a phenomenon where the force in a bolt in a hanger-type connection is increased by the development of a prying force, $Q$. Several combinations of bolt capacity and plate thickness can satisfy equations and static requirements.

A new philosophy of material tear-out of tension splices is also included. In the existing *Structural Steel Detailing*, material was checked by equating the allowable shear stress of the material to the weld capacity. Thus $0.4F_y = 0.707\ (0.3)F_u w$. For $F_y = 36$ ksi and $F_u = 70$ ksi, the minimum plate thickness is $t = 1.03w$, where $w$ is the length of the weld leg. The new procedure recognizes that material tear-out is more of a block shear model, and new rules for this are included.

**Chapter 2:** "Metallurgy and Welding" contains practically all the information required by the structural engineer on this subject. Topics include: weldability, welding processes, types of welds, welded beam-to-column connections, electrode and process nomenclature, nondestructive testing (NDT) methods including ultrasonic and radiographic, fracture control and lamellar tearing.

The factors which affect weldability, chemical composition, geometric properties and grain size are discussed. The advantages and disadvantages of various welding processes are outlined to help the designer and fabricator with shrinkage and distortion control problems, and the text suggests ways to minimize lamellar tearing.

**Chapter 3:** "Simple (Type 2) Connections" treats the behavior and design of all popular connections—framing angles, shear tabs, end plates, single angles and tees. The chapter examines each possible failure mode and presents rules to prevent them.

For bolted connections, this includes bolt shear, material net shear, web tear-out (block shear) and the effect of edge and end distance on bolt bearing capacity. For welded connections, new material is presented for block shear on coped beams.

The question of eccentricities on bolted connections often arises. For one-sided connections, eccentricities on the outstanding legs should be accounted for. The 8th Edition Manual Tables (Part 4) account for them. For a single vertical row on the beam web, eccentricities have been ignored traditionally, and no distress has ever been reported. For two vertical rows on the web, however, less was known at the time both the 8th Edition Manual and this new text were published, so no procedure is included in either book. The author recommends Joseph A. Yura's suggested block shear model published in the January 1983 *Journal of the Structural Division* of ASCE.

Single-plate shear connections are a relatively new Type 2 simple connection. Although it was in use for years, it was only in 1980 that Prof. Ralph M. Richard of the University of Arizona provided an analytical procedure for design, which is included in this book.

Figure 1 shows a typical shear tab connection. It has some inherent stiffness, and improper design might disqualify it as a Type 2 connection. Also, the rigidity might impose force on the top part of the weld that connects the plate to the abutment. In the past, many connections have been designed with a moment equal to the reaction times the distance $a$.

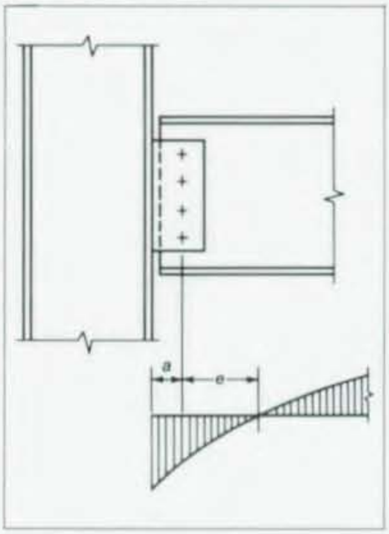



Figure 1. Typical shear tab connection.

Richard's research indicates that, for design purposes, the moment should be calculated by multiplying the reaction by $a + e$ ($e$ is determined as a function of connection dimensions and beam properties. Using an ultimate design procedure, the plate thickness and weld size is determined.

**Chapter 4:** "Moment Connections" includes the design and detailing of welded beam-to-column connections, end plates, flange plates, cap and seat angles and flexible wind connections. The text emphasizes that, with a welded beam-to-column connection, the web of the beam need not be welded directly. A better method is to weld a shear tab to the columns and bolt the beam web to the shear tab. Flanges of the beam are then field-welded. This connection has been tested thoroughly at Lehigh University and the University of California/Berkeley. Those tests have determined the connection can easily achieve the plastic moment of the beam.

The procedure for designing the detailing end-plate connections was developed by Krishnamurthy, as presented in the 8th Edition Manual. The procedure is empirical, and is based on physical tests and a finite element analysis. The result is a method to determine bolt size and end-plate thickness. However, no method to determine column stiffener requirements was included. Stiffener requirements for end-plate connections is the subject of a major research project at the University of Oklahoma under the direction of Thomas Murray. At publication time of *Engineering for Steel Construction*, research was not complete. But, sufficient progress has been made to recommend these tentative, conservative rules.

1. In the region of the beam compression, flange stiffeners are not required when the column web $t_w$ exceeds:

$$t_w \leq \frac{P_{bf}}{F_y\,[t_b + 5k + 2t_p + 2w]}$$

where $P_{bf}$ = factored beam flange force, kips; $F_y$ = specified yield strength of the columns, ksi; $t_b$ = beam flange thickness, in.; $k$ = distance from outer face of flange to web toe of fillet of column section, in.; $t_p$ = end-plate thickness, in.; $w$ = end-plate fillet weld leg dimension, in. It is anticipated that final research will permit $5k$ to increase to $6k$ or even $7k$. However, until then, $5k$ is recommended.

2. In the tension region, a rule of thumb is proposed until research is completed. This rule is conservative, and states that if the column flange thickness is as thick as the bolt diameter (determined by the Krishnamurthy, or Manual, procedure), then stiffeners are not required. Otherwise, they are.

A welded beam connection into the weak axis of a column has to be designed and detailed with special care. Research is presently underway at Lehigh University to determine how this connection can best be designed to insure it will behave in a ductile fashion once the plastic moment of the beam is attained. Preliminary tests indicate that ductility could be impaired if the flange connection plates are terminated at the toes of the column flange. Although final results are not yet in, several recommendations have been made to improve the connections:

1. Use connection plates slightly thicker than the beam flange thickness.
2. Use backup stiffener.
3. Extend the connection plate beyond the tips of column flanges.

Flexible moment connections designed to carry only the wind moment are also treated in this new AISC text. Several types discussed include cap and seat angles and flange plates.

**Chapter 5:** "Skewed, Sloped and Canted Beam Connections" is an update of a similar chapter in the 2nd Edition *Structural Steel Detailing*. It will be of more interest to the detailer than the designer. However, the designer should be aware of the various eccentricities sometimes associated with these connections. One change from previous detailing practice is that all working points will be located on the material. Previously, they could be located in space.

**Chapter 6:** "Columns" includes detailing procedures, stiffener requirements and design, built-up members, lifting lugs, splices and base plates. Column stiffeners are required because of a local deficiency in column web or column flange thickness. In the area of beam compression, flange stiffeners might be required to prevent column web yielding or buckling. In the tension area, stiffeners prevent web yielding and promote a uniform stress distribution to the column flange. If a non-uniform flange distribution exists, the beam-to-column flange weld could be overstressed. The text notes that determination of column web stiffeners should be the responsibility of the designer, rather than of the detailer. The reason is that, for economy, the designer might prefer a column with a larger web or flange to avoid adding stiffeners.

As in the 2nd Edition of *Structural Steel Detailing*, the new text includes details of recommended column splices. Several are shown, but the one which may need special attention is the direct field-welded splice. A clean detail with no extra pieces, it is economical to prepare in the shop, and uses minimal field labor.

This chapter also discusses the design of column base plates. The procedure in the 8th Edition Manual is recognized as ultra-conservative in the case of small base plates—those large enough in plan to include just the section profile. The procedure in *Engineering for Steel Construction*, based on a yield line theory developed by R. S. Fling, results in reasonably sized base plates. However, since the material included in the book, a third method, developed by Thomas Murray of Oklahoma University, has been proposed. This method was published in AISC's *Engineering Journal*, 4th Quarter 1983. Relatively simple, it is expected to be very popular.

**Chapter 7:** "Framing for Heavy Construction" includes bracing, crane girders, trusses and heavy bracing connections. Except for the material on heavy bracing connections, it is primarily an update of that in *Structural Steel Detailing*. A heavy bracing connection is defined as a connection involving a column, beam and diagonal brace. Usually, very heavy loads are involved, such as those expected in power plants, industrial structures and high-rise buildings.

Designers differ widely in the models they use to design these connections. For this reason, AISC has underway a major research project at the University of Arizona, which should be completed in a year or so. In the meantime, William A. Thornton of Cives Corporation has developed a procedure that is included in this new text. □

Modern Steel

Construction

Readership

Survey

Please check and mail.

No postage necessary.

# *Steel Construction* Readership Survey

Please check and mail. No postage necessary.

ɜs ☐ No ☐

ɜs your firm? Check
Steel Fabricator 6. ☐
Supplier to Industry 7. ☐
Government Agency 8. ☐
DOT 9. ☐
Educator 10. ☐

ɜs your function? Check
1. ☐
2. ☐
3. ☐
4. ☐
5. ☐
6. ☐

*struction*, do you mostly:

*Modern Steel Construction* of
No ☐

ations do you read regularly?
1. ☐ ______
2. ☐ ______
3. ☐ ______
4. ☐ ______
5. ☐ ______
6. ☐ ______
7. ☐ ______
8. ☐ ______

vould you like to read in MSC?

______

______

our firm primarily engaged?
______%
______%
______%
______%
______%
______%
______%
______%
______%
______%
______%
100%

**2** Is MSC circulated in your office? Yes ☐ No ☐
How many people read it? ______

**4** *Approximately how many are in your firm? Check*
Less than 10 ☐
10-20 ☐
20-30 ☐
30-60 ☐
100-300 ☐
300 or more ☐

**6** Are you directly responsible for specifying the type of framing used for the projects designed by your firm?
1. Yes ☐ 2. No ☐

**8** Do you read (scan)
1. every issue ☐
2. most issues ☐
3. some issues ☐
4. no issues ☐

**10** Are the articles technical enough?
1. Yes ☐
2. No ☐
Too technical? 1. Yes ☐ 2. No ☐

In structural engineering design area, how do you compare MSC with others you read? Check

| Better than | As good as | Not as good as | Can't compare |
|---|---|---|---|
| ☐ ______ | ☐ ______ | ☐ ______ | ☐ ______ |
| ☐ ______ | ☐ ______ | ☐ ______ | ☐ ______ |
| ☐ ______ | ☐ ______ | ☐ ______ | ☐ ______ |
| ☐ ______ | ☐ ______ | ☐ ______ | ☐ ______ |
| ☐ ______ | ☐ ______ | ☐ ______ | ☐ ______ |
| ☐ ______ | ☐ ______ | ☐ ______ | ☐ ______ |
| ☐ ______ | ☐ ______ | ☐ ______ | ☐ ______ |
| ☐ ______ | ☐ ______ | ☐ ______ | ☐ ______ |

**14** Of the buildings your firm designs, what percentage are
steel-framed ______%
concrete-framed ______%
100%

**15** Of the bridges your firm designs, what percentage are
steel ______%
concrete ______%
100%

**16** Are you a member of AISC?
1. No ☐
2. Yes ☐
Which type? Active Member 1. ☐
Associate Member 2. ☐
Professional Member 3. ☐

**17** Do you have any other comments on *Modern Steel Construction* or American Institute of Steel Construction you'd like to give us?

______

______

Fold & Staple

NO POSTAGE
NECESSARY
IF MAILED
IN THE
UNITED STATES

BUSINESS REPLY MAIL
FIRST CLASS PERMIT NO. 11588 CHICAGO, IL

—POSTAGE WILL BE PAID BY—

AMERICAN INSTITUTE OF STEEL CONSTRUCTION
400 North Michigan Ave.
Chicago, Illinois 60611

# 333 Wacker Drive: Building with a Historical Flourish

**by Robert L. Miller**

*Robert L. Miller* was vice president of GCE of Illinois, consulting engineers, when the 333 Wacker Building was under construction.

Three centuries ago, the French explorer LaSalle steered his canoe past the site of the new 333 Wacker Building in Chicago's near Loop area.

On a momentous day in March 1982, a pompously costumed "LaSalle," portrayed by a local educator-adventurer, landed his 20-ft canoe at the building site. His arrival marked the beginning of topping out ceremonies with local dignitaries, owners and contractors that ended with the hoisting of a commemorative plaque to the building's highest point, signaling completion of the steel framework.

*The ambition of every participant in a* building development—owner to tenant, architect and engineer to contractor—is to be involved in outstanding projects. The design disciplines hunger for projects that present challenges to stir our imagination, but not just as a mental exercise. The fruits of our imaginative efforts should be vital to the success of the project. Chicago's new 36-story, 333 West Wacker Drive Building is an excellent example of this type of project.

The triangular site presented the architects with unique challenges and opportunities. Their labors culminated in a wing-shaped, 36-story office tower that incorporates a curved face along Wacker Drive. The curved facade flows naturally with a bend in the Chicago River that turns in front of the building. In contrast to this magnificently curved surface, the three remaining sides are straight, but they have notches that break up the flat surfaces.

While the triangular site presented an interesting challenge to the architects to come up with a working footprint of the building, the constraints below the ground presented the structural engineers with their first challenge. Investigation of the subsurface indicated a caisson foundation system would be most applicable. However, easements from two subway lines and the close proximity of the foundation to the property line loomed as formidable challenges.

Concessions by the Chicago Transit Authority eliminated one easement. Grade beams span to, and cantilever off the caissons to support the structure along both the Lake Street subway easement and the Wacker Drive foundations.

The shape of the building limited the number of available interior columns, and spans for the gravity system varied widely. Any structural system that relies on continuity or repetition could not be incorporated economically in this project. In addition, the curved surface and notches presented additional engineering challenges.

Engineers analyzed five gravity and lateral framing systems. These analyses included four steel schemes consisting of short-span composite beams and girders; stub girders with composite beams; and long span composite beams supported by girders on the short span. The lone concrete system analyzed was concrete pan joists supported on haunched girders. Since concrete relies predominantly on continuity and repetitiveness for its economy, and steel does not, the engineers concentrated most of their efforts on steel.

With a floor-to-floor height of 12 ft-2 in., and a ceiling height of 8 ft-6 in., the proposed framing systems had to provide shallow secondary framing members and limited penetration through primary members. With this in mind, along with cost and architectural analysis, the steel framing system—incorporating long-span beams, with girders framing in the shorter direction—was selected.

**Modification Cost Minimal**

Architecturally, this structural scheme was attractive, since it provided for a limited number of interior columns in tenant spaces. In the low-rise floors, only two columns are within leasable areas. Additionally, this scheme offers flexibility to accommodate special requests for present and future tenants. As tenants request increases in the live-load capacity of their floors, the capacity of floor members can be increased by welding plates and structural tees to the bottom flanges. The structure was also modified to include interior stairs for two tenants after building erection had begun. The cost of these modifications was a fraction of what it would have been if a concrete structure had been built. One inherent potential problem of dead-load deflection in the long-span beams was dealt with by cambering those members. Shoring was considered, but the general contractor found cambering to be about one half the cost.

The floor framing system consists of 3¼-in. lightweight concrete slabs over 2-in. metal deck on 16-in. deep cambered composite beams, 10 ft o.c., spanning predominantly between 40 and 45 ft; 24-in. and 30-in. deep composite girders spanning 30 to 45 ft with a limited number of these girders having reinforced penetrations for mechanical ducts. Two thirds of the floor framing systems were designed using high strength (50,000 psi) steel. This system resulted in a steel weight of approximately 17 lbs./sq ft, including the lateral system. Typical office buildings in Chicago usually range from 14 to 16 lbs./sq ft. Since 333 West Wacker Drive is hardly typical, the engineers were satisfied that the final steel weight was the lowest limit for this building.

The elements discussed so far may be interesting, and certainly challenging. However, as the architectural design evolved into an outstanding architectural addition to the Chicago building community, the engineers' imagination would have to become part of the design effort.

**Extensive Analysis Required**

The unique shape of the tower required extensive analysis to determine the impact of wind forces on the building. A wind tunnel test was essential to derive an effective, economical wind system. Dr. Isyumov of the University of Western Ontario conducted the wind tunnel tests. Results provided the data required to decide on both the most applicable wind system, and the

Last steel up for 333 Wacker Building at historic site (l.). Above (and r.) shows building's structural steel frame as it curves to adapt to triangular site. Renowned Merchandise Mart in background (r.).

History is reenacted as "La-Salle's" canoe heads for shore at river juncture (above). "LaSalle" then exchanges greetings with local dignitaries prior to topping out ceremonies.

analysis of that system.

The wind tunnel data indicated significant torsional loads would be applied to the lateral structural system. The effective eccentricity of the wind load on the curved surface was about 60 ft. To resist this torsion, a moment-connected exterior frame concealed behind the curtain wall would have been ideal. This lateral system would have been attractive, although highlighting the structure of the building was not an initial architectural consideration. However, since the sides of the building are not perpendicular to each other, transferring loads around the corners efficiently would be impossible using moment connections. In addition, the number of available exterior columns was limited by architectural considerations. Without tightly spaced columns, the rigid frame lacked economical stiffness. Finally, notches in the side and back walls would interrupt the continuity required for rigid frames.

To determine if the centrally loaded core could be of assistance in resisting applied torsional loads, core framing in both steel and concrete was analyzed. The determination was it would be more efficient and economical to incorporate an exterior steel frame as typical and provide a stiffened concrete core only where the exterior frame could not be used.

An X-braced external system with limited horizontal projection on the curved surface, running the full width on the remaining three sides, was proposed. This system was found most efficient in meeting the torsional challenge. The bracing is designed on 12-story modules. With this module, diagonals interact column/beam nodes at every second floor. At the notches, links are incorporated to transfer axial loads from one truss node to another. Zoning requirements for the building resulted in a plaza design that interrupted this system and provided us with another challenge.

**Yet More Challenges!**

By recessing the building enclosure at the ground floor, the zoning ordinance provides for a five-story bonus. The arcade thus created made it impossible to extend the X-brace system to the foundation. To work around this, the engineers used the floor diaphragms, beginning at the third floor, to transfer the load to the core.

Analysis indicated the majority of the load transfer would take place at the third floor. By increasing the third floor slab thickness, employing extensive reinforcement on that level, and by providing sufficient shear studs on the remaining transfer floors, the marriage of the exterior X-brace system and the concrete floor diaphragms was complete. These floor diaphragms were then anchored to the core primarily by using shear studs. The third floor load transfer incorporated, in addition to shear studs, drag beams and reinforcing bars anchored directly to the top of the concrete core. Designing the core walls in concrete from the caissons up to the third floor permitted stiffening of the core sufficiently to resist total torsional loads. Once these loads were in the core, the concrete slab at the ground floor and the slab on grade (as diaphragms) deliver the forces to the foundation system. Special X-bracing was employed in the core above the third floor to help control the total horizontal building drift.

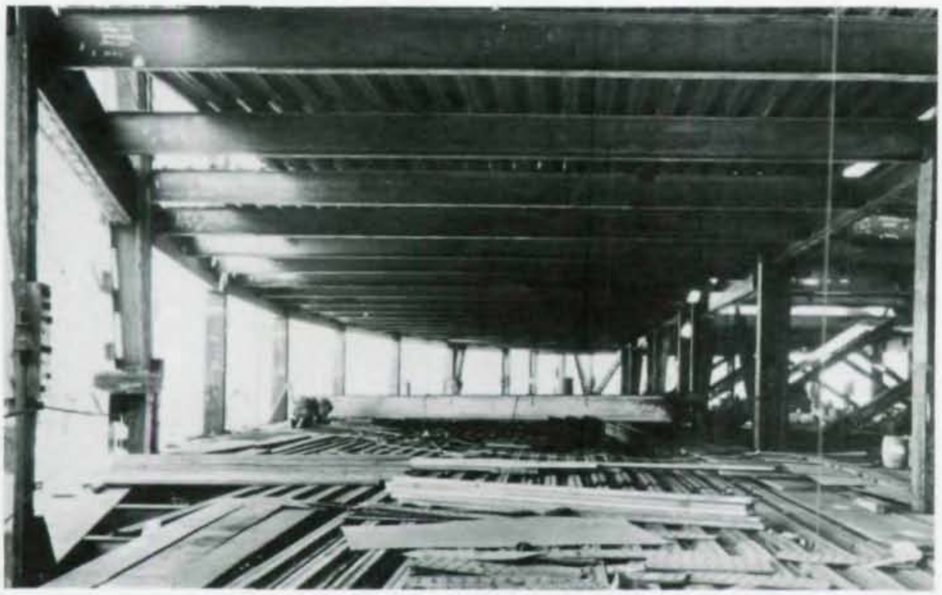
When the general contractor analyzed the construction schedule, he decided that time could be saved by revising the normal sequence of construction. This presented a unique and unexpected challenge.

Design and documentation was based on the concrete structural system extending up to the third floor at the core and to the ground floor for the balance of the building. From these points, the steel structure would have originated. Due to the contractor's concern that the complicated lower level concrete work would not be completed in time for the scheduled beginning of steel erection, erection was started at the top of the caissons. This approach required incorporation of extensive temporary support systems below the third floor. This temporary steel, encased in concrete, remains embedded in the final structure. This temporary system permitted steel erection through the 13th level. With the final concrete structure in place, steel erection above the 13th level proceeded unhindered.

With this unique approach to construction, extensive modifications were required in the concrete structure so that reinforcing steel would be placed in a manner consistent with the design. The result of the design and construction team effort has already been recognized by an award from the Structural Engineers Association of Illinois for its "contribution to the state of the art of the structural engineering profession."

When the joint venture of Urban Investment and Development Co. and Equitable Life Assurance society of the U.S. decided to develop this site they knew it would be a unique building. The result of the efforts of the entire building team—a building whose uniqueness will never be duplicated! □

Under-the-deck photo shows curved structural steel frame.



**Architects**
Kohn, Pedersen, Fox, New York City; and
Perkins & Will, Chicago, Illinois

**Structural Engineer**
GCE of Illinois, Inc.
Chicago, Illinois

**General Contractor**
Inland Construction Co.
Chicago, Illinois

**Steel Erector**
American Bridge Division, U.S. Steel Corp.

**Owners**
Urban Investment and Development Co., and
Equitable Life Assurance Society
Chicago, Illinois

# Make sure your bolts are properly tensioned. Specify Coronet Load Indicators.

**You don't calculate your building connections by guesswork. So why allow guesswork to determine how well bolts are tightened in the field?**

That's exactly what happens when high-strength structural bolts are installed using the "turn-of-nut" method. This method depends on guesswork because it depends on where nut rotation is started and how it's measured. Different rotation requirements are necessary for different bolt lengths. And nut rotation can collect ply compression instead of bolt tension.

Compounding the unreliability of this method of bolt installation, there is no proof that the installed bolts have been tightened properly. Inspection is performed with a calibrated torque wrench which does not necessarily measure tension.

With Coronet Load Indicators, however, you can be sure structural joints are properly assembled. As the bolt is tensioned, the clamping force flattens the protrusions, reducing the gap.

Coronet Load Indicators are accurate because the specified gap indicates correct clamping force, or tension. They provide immediate visual proof that 100 percent of the bolts have been correctly tensioned. They save costs because they are simple and easy to install with standard socket wrenches, and inspection is considerably faster than with a calibrated torque wrench. And Coronet Load Indicators can help avoid costly callbacks which lead to schedule disruptions and loss of productivity.

So, know bolts have been tensioned properly by eliminating the guesswork and save the after costs of loose bolts, too. Specify Coronet Load Indicators. Write or call today for an up-to-date fact file.

The new Columbia Center in Seattle, Washington will be 76 stories when completed, making it the tallest building west of the Mississippi. *Owner & Developer:* Martin Selig; *Structural Engineer:* Skilling, Ward, Rogers, Barkshire; *General Contractor:* Howard S. Wright & Company; *Steel Fabricator:* Samsung Corporation; *Steel Erector:* American Bridge; Division of U. S. Steel.

**Cooper & Turner Inc**

**Coronet Load Indicators**
522 Parkway View Drive
Pittsburgh, PA 15205
Telephone (412) 787-2253 · Telex 812381

Glynwed International

# Cedars-Sinai Medical Center: Creating Space Where There is None

**by Kenneth Liu**
**and**
**Michael L. Bobrow**

*Kenneth Liu*, AIA, is principal, architecture, and *Michael L. Bobrow*, AIA, is a partner in the architectural firm of Bobrow/Thomas and Associates, Los Angeles, California.

The Cedars-Sinai Medical Center in Los Angeles is one of the most modern hospitals in the nation. When it was constructed in 1976, every effort was made to provide for future requirements, as well as the current needs of the facility. But to foresee all future requirements, particularly for a medical facility, is impossible.

A few years later, the hospital needed additional space for a conference center—a small auditorium and several rooms in which to hold seminars. The architect, together with the structural engineer, came up with a plan for a 500-seat auditorium, flexible space for six seminar rooms, and even a small art gallery.

The first problem the architect faced was to find a spot for the facility in the confined, built-up property of the hospital. The answer was the east plaza level of the center, a landscaped area on the roof of the parking structure of the eight-story center. This plaza, while attractive, was not fully utilized, and it would provide a convenient location for the addition. The primary constraint as far as the structural engineer was concerned was weight. The plaza was not designed to support another structure. If it were necessary to strengthen the existing structure below the plaza level, the new facility would not be economically feasible. Part of the solution,

in addition to clearing the area, was to remove some of the planters and benches from the surrounding area. The rest of the solution was to provide a light, efficient structural steel frame for the addition within the constraints of the seismic codes and also to carefully monitor the weight of other building materials.

The architectural challenge was to create a structure which would appear as a natural extension of the existing medical center, yet maintain its own individual identity. Continuity was achieved with the same solar-bronze glass as the existing building. Individuality was expressed by breaking down the scale of the building

Cedars-Sinai Medical Center, Los Angeles, acquired needed space for conference/auditorium/seminar rooms on plaza above parking deck (l.). Below, ample conference area and large auditorium accommodate hospital's varied needs.

by employing setbacks along the roof plane. The resultant crystalline structure, through the play of the same color and a radically different scale, permits the addition to visually bounce back and forth between its own identity and that as an extension of the main building.

Internally, arrival in the conference center is defined by a two-story lobby gallery, which creates its own sense of special place within the center. This central area, and the one-story wings on each side, accommodate a 6,000-sq ft auditorium which is divisible into six rooms designed for seminars. Advanced, state-of-the-art audio-visual equipment is available in all meeting areas. In addition, there is a permanent art gallery. In its more than 1,400-sq ft of gallery and lobby space, changing exhibitions of the hospital's highly esteemed contemporary art collection are held.

Technically, what is particularly unusual about the building is the use of the plaza level as its site. The design team, working with the structural engineer, developed a light structure that would meet the seismic codes. The $3.5-million addition, named the Harvey S. Morse Conference Center for the Los Angeles philanthropist, provides 12,000 sq ft of space for medical professionals, hospital staff and community use. □

**Architect**
Bobrow/Thomas and Associates
Los Angeles, California

**Structural Engineer**
Albert G. Presky and Associates
Los Angeles, California

**General Contractor**
Jones Brothers Construction Co.
Los Angeles, California

**Steel Fabricator**
Riverside Steel Construction
Santa Fe Springs, California

**Owner**
Cedars-Sinai Medical Center
Los Angeles, California

# Prestonwood Baptist Church: Steel Wins the Roof "Sweeps"

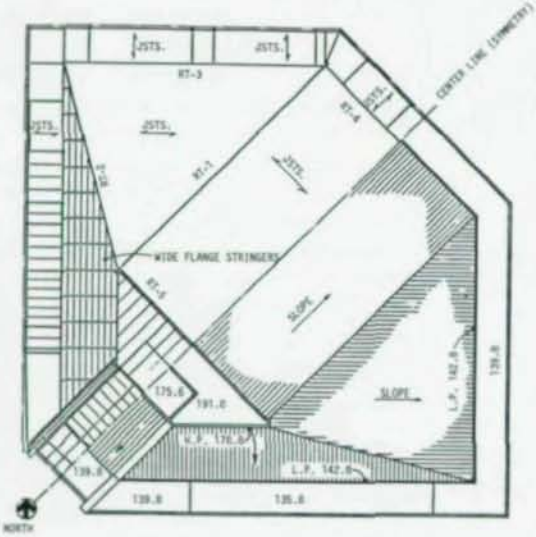
The congregation of Prestonwood Baptist Church in Dallas required a 4,000-seat auditorium where the whole congregation could not only observe but also be a part of the proceedings. A large area above the pulpit was desired to provide a sweeping view of the choir; to house audio equipment and speaker cluster in the ceiling and the organ pipes behind the choir platform, both for proper acoustics; and to provide uninterrupted sightlines to the pulpit from any part of the auditorium so that this large congregation could feel close.

The roof sweeps up to focus on the cross. A copper batten seam roof, a time-honored element in church architecture, was used on the sloping roof. The roof was shop-fabricated and delivered to the site without serious delay in erection time. Figure 1 shows the roof framing plan and roof slopes. The roof trusses RT-1 and RT-2 define the slope and serve as ridge trusses. Deep long-span joists of W12 sections in top and bottom chords spanned the trusses. Web members were either W12 sections or double angles. The top chord was oriented with a W12 web vertical. In the bottom chord, the W12 web was oriented horizontally, so the bottom chord bracing could be at large distances (Photo 1).

These trusses were shipped in two sections and assembled on the ground before being hoisted into position. The functional requirement of unobstructed vision in the auditorium dictated that the ends of these trusses be supported at different heights (also Photo 1). The column height at the high end of truss RT-1 was 77 ft-2 in., of which 33 ft-4 in. was unbraced by any intermediate level framing. Maximum truss spans are 131 ft-6 in.

At the top of this column it was necessary to support several trusses—RT-1, RT-2, RT-3 etc.—at different levels (Photo 2). To accommodate all these trusses without serious eccentricity and to provide for lateral rigidity for wind loads along the RT-1 direction, W33 rolled shapes, more economical than built-up shapes, were used as columns. For the length where the weak axis slenderness was large, this W33 section was stiffened by two WT7 sections welded to the web so the column was in the shape of two intersecting wide-flange shapes. As seen in Fig. 1, on the west and south sides of the roof the slopes were very steep. In these faces, W8 sections were used as stringers in lieu of open web joists so that biaxial moments could be resisted satisfactorily (Photo 3). □

Exterior of Prestonwood Baptist Church, Dallas (l.) shows steel-framed roof sweeping upward to cross. Long truss spans in interior (r.) provide uninterrupted sightlines for worshippers.



**Architect**
JPJ Architects, Inc.
Dallas, Texas

**Structural Engineer**
Mullen & Powell, Inc.
Dallas, Texas

**General Contractor**
Robert E. McKee, Inc.
Dallas, Texas

**Steel Fabricator**
Austin Steel Company, Inc.
Dallas, Texas

**Owner**
Prestonwood Baptist Church
Dallas, Texas

Figure 1. North half of roof plan shows structural steel framing. Sloping roof is identified by high point (H.P.) and low point (L.P.) elevations.

Photo 1 (above) is structural steel frame of roof. Long spans (up to 131 ft-6 in.) provide column-free areas for large auditorium. Photo 3 (above, l.) is exterior of steel frame, shows steep roof slopes at south and west faces.

Photo 2 (l.). Tall column supports trusses RT-1, RT-2. Notice weak axis stiffening to bottom of trusses.

# Welded Steel Duct Designed to Bridge Factory Buildings

**by German Gurfinkel**

Transfer of exhaust gases from the top of a smelter building in the new Florida Steel Company mill in Jackson, Tenn. into cleanup equipment on the ground was necessary to collect dust and filter the air before releasing it into the atmosphere. Design of a 322-ft long 12½-ft dia. duct, inclined 15° with the horizontal, was required.

Conventional structural designs required support for the duct all along its length in various ways. These were not as competitive in cost, and time of fabrication and erection, as one design which used the duct itself (in bridge-type fashion) to support the loads (see Fig. 1).

*German Gurfinkel* is professor of civil engineering, University of Illinois-Champaign-Urbana, Illinois.

Building layout at the mill permitted placement of three supporting towers for the inclined duct. Thus, design of the duct as a two-span continuous bridge with two end cantilevers became possible (Fig. 2). The maximum span between towers, and the longest cantilever, were 143-ft and 55-ft long, respectively.

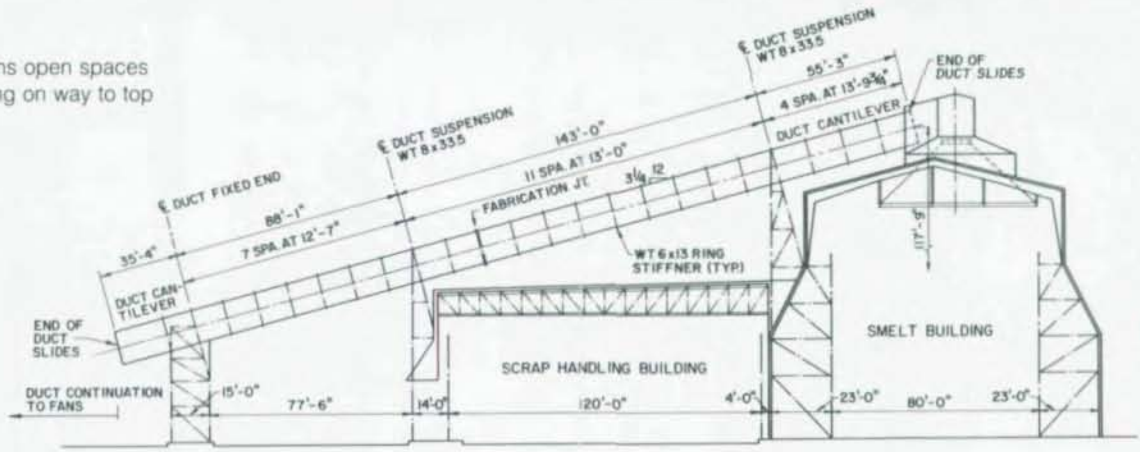
Temperature effects governed design of the duct supports at the towers. Exhaust gases of 400° F keep the duct quite hot during normal plant operation. However, a potential shutdown of operations during the winter could possibly bring the temperature of the duct down to 0° F. Thus, expansion of the duct between a possible 70° F at erection time and 400° F during service, followed by contraction to 0° F, had to be accommodated by the supports.

This was accomplished by rigidly attaching the duct only to the lower tower, while suspending it, hammock-type fashion, from the other two towers. Thus, the duct expands and contracts about the lower-tower support. Total longitudinal displacement of the duct end at the smelter building, about the lower-tower support, was calculated at 9 in. Similar displacements at the two intermediate tower supports were calculated at 7¼ in. and 2¾ in., respectively. Since these displacements can easily be realized by the long suspension rods, temperature-induced stresses are hardly generated in the duct.

Thus freed from temperature-induced stresses, design of the duct was governed by gravity loads and wind. Use of a stiffened-shell cross section required only a 3/16-in. thick plate reinforced by a set of longitudinal and circumferential stiffeners

Figure 1. Welded steel duct (322 ft long x 12.5 ft dia.) bridges three supporting towers. Duct permits removal of 400° F exhaust gases from top of smelter building.

Figure 2. Duct bridge spans open spaces and scrap-handling building on way to top of smelter.




SCHEMATIC ELEVATION

(see Fig. 3). ASTM A36 steel was specified and the structure was welded together with E6018 electrodes.

Weight of the stiffened shell was calculated at 505 lb./ft. Provisions were made also for future placement of longitudinal grating and railing, at an additional 95 lb./ft, to permit easier access and inspection of the duct. Thus the total gravity load was taken at 600 lb./ft.

The live load on the duct, consisting of snow and ice, was calculated at 330 lb./ft. Wind load was estimated at 30 psf (for a 100-year mean recurrence interval, a basic speed of 80 mph and exposure Type C). This led to a uniformly distributed wind load of 375 lb./ft acting against the duct in a plane inclined 15° to the horizontal and containing the longitudinal axis of the duct.

Various loading conditions were considered in design of the duct. It was determined that the governing load was caused by the combined action of self weight plus snow and ice. Design of the cross section of the duct called for providing strength to resist the simultaneous effects of a 1,580-kip-ft bending moment, a 15-kip longitudinal thrust and a 71-kip transverse shear.

Design criteria called for a shell stiffened externally by a set of 20 continuous longitudinal angles. The maximum compressive stress, due to combined action of bending moment and thrust, was 8 ksi on the critical angle stiffener, acting compositely with 10 in. of effective flange from the shell. Transverse circumferential stiffeners were designed to prevent buckling of the longitudinal stiffeners. Their spacing was calculated to allow as much as one percent deviation in the longitudinal axis of

Figure 3. Front view of duct as it hangs hammock-style from 2½-in. dia. steel bars attached to top beam of supporting tower. Duct is welded cylindrical shell 3/16-in. thick, stiffened longitudinally by 20 3x3x⅜-in. steel angles. Transverse stiffening is WT6x13 rings 13 ft apart. At support towers, WT8x33 ring supports are used.

the duct between them. This limitation is considered well within normal fabrication tolerance and is unlikely to be exceeded.

A cross section of the duct at one of two suspended supports is shown in Fig. 5. Note the 3/16-in. thick steel shell stiffened by 20 longitudinal 3½ x 3½ x 3/8-in. steel angles. In addition, there is a WT8x33.5 circumferential stiffener attached to the duct that is used to transfer the bridge reaction through two 2½-in dia. hanger rods to the supporting-tower frame.

The duct is provided with resistance to wind-induced forces at each supporting tower. The keel-type devices used at the intermediate and upper towers, where the duct is suspended by hanger rods, may be of special interest (see Fig. 5). Thus, in the same fashion as a sailboat is stabilized against the wind, the duct is provided at each tower with keel-type devices, one below and one above. These devices are activated in the wind by bearing against the transverse beams of the supporting frames. Longitudinal displacements of the duct at these locations are facilitated by sandwiched neoprene plates.

The duct was welded together in the shop and fabricated in longitudinal sections for shipment. These were temporarily bolted together in the field, until welded permanently in place. The finished duct-bridge has been in continuous operation since late 1981. □

**Architect** (mill)
TLM Engineers
Jackson, Tennessee

**Structural Engineer** (ducting)
Professor German Gurfinkel
University of Illinois-Champaign-Urbana

**Owner**
Florida Steel Company
Jackson, Tennessee

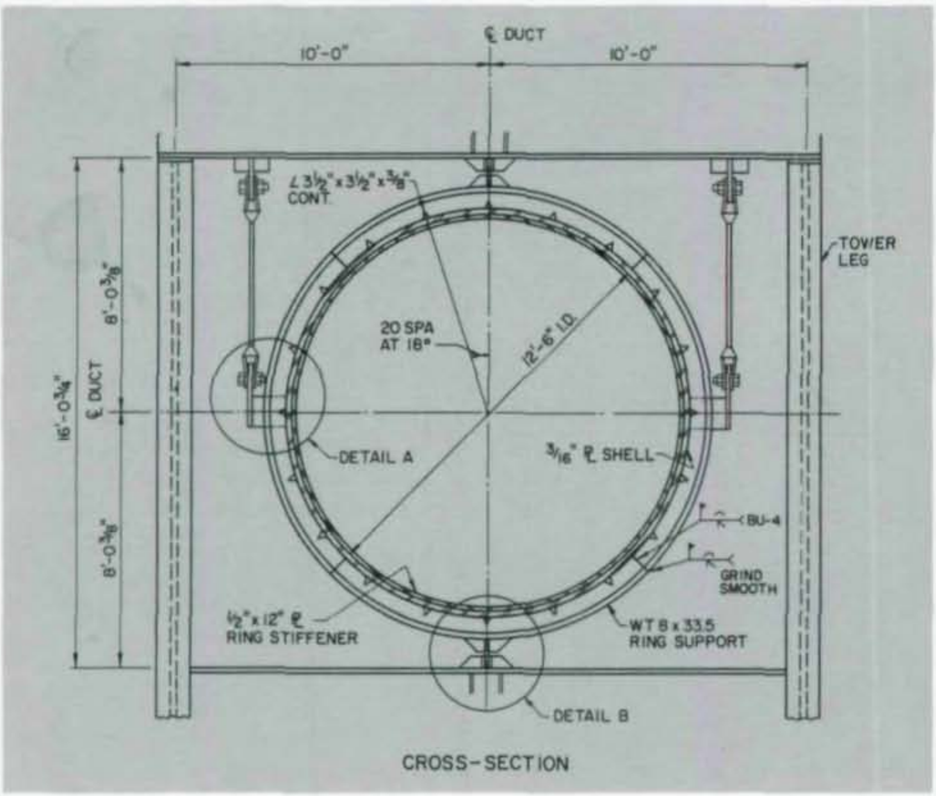



Figure 4. Cross-section of duct bridge at a suspended support.

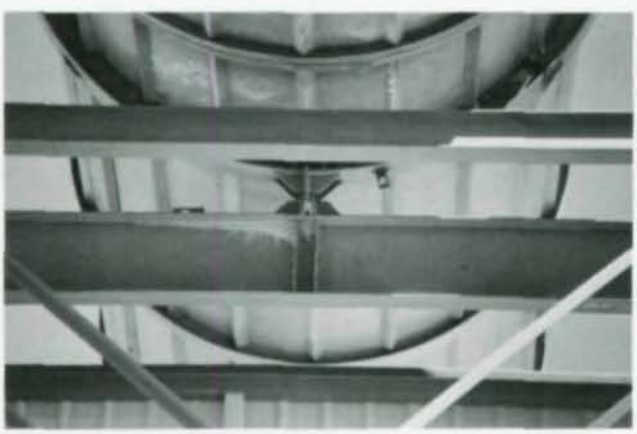
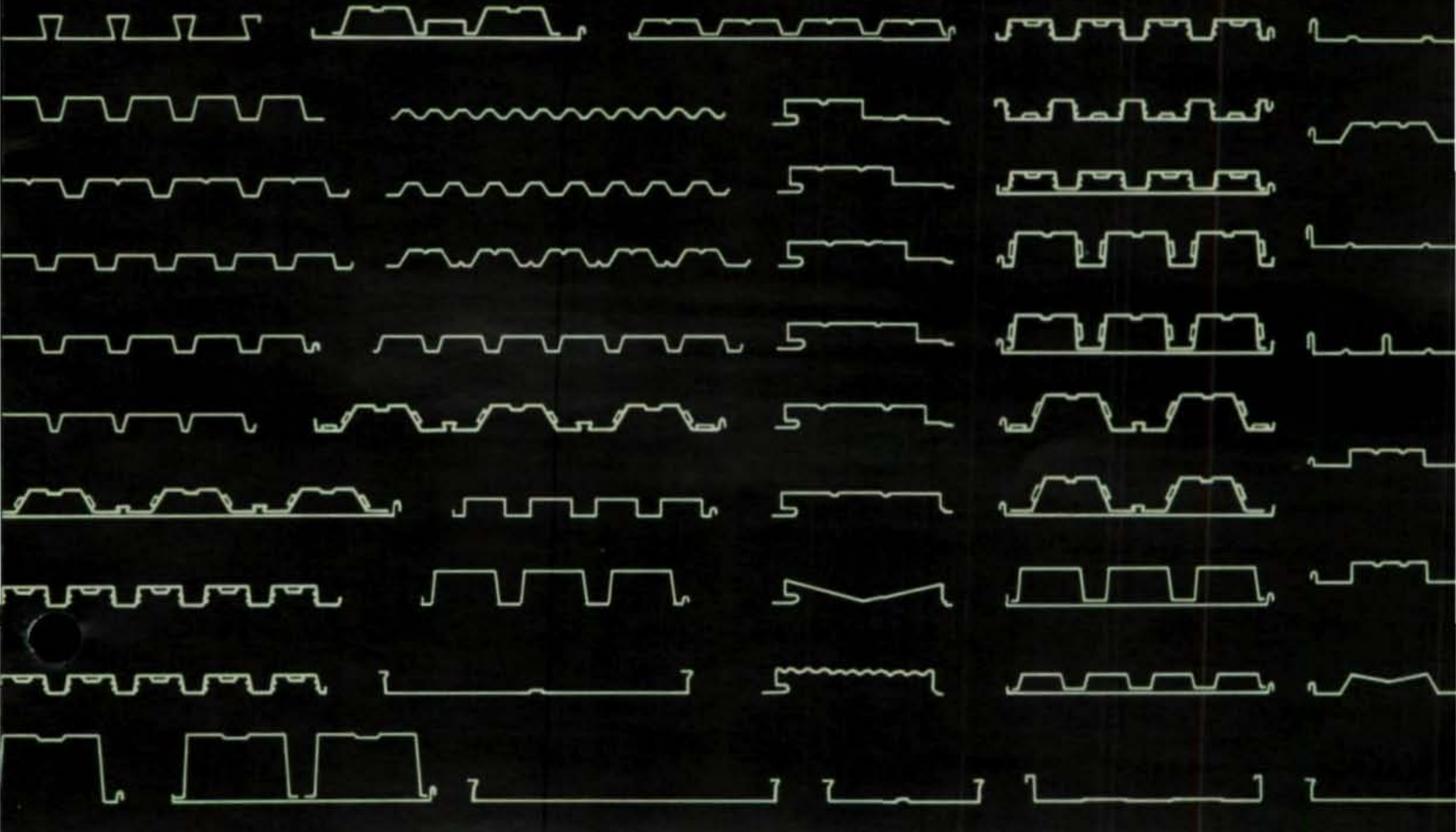


Figure 5. Note keel-type device on duct underbelly that provides lateral restraint without hindering longitudinal displacements. Built in pairs, these elements are effective in resisting wind-induced lateral forces that act against bridge.



**AMERICAN INSTITUTE OF STEEL CONSTRUCTION**
**The Wrigley Building, 400 North Michigan Avenue**
**Chicago, IL 60611**

*Address Correction Requested*

BULK RATE
U.S. POSTAGE
PAID
WHEATON, ILL.
Permit No. 1786