A9.25-1

VOLUME XXV NUMBER 1/FIRST QUARTER 1985

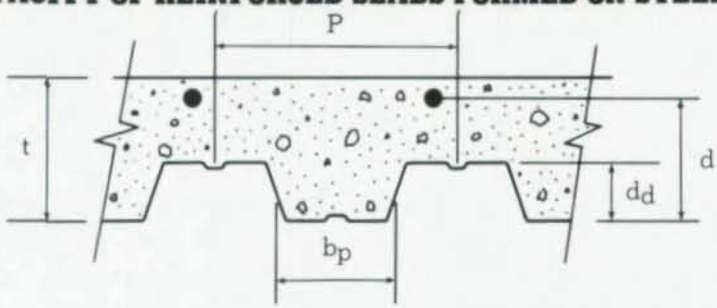
MODERN STEEL CONSTRUCTION

Exchanging Times in Chicago's South Loop
The Leading Edge in Industry Trends
The Symphony of Steel's Flexibility Continues
A Fast Track for a Fast Track
1984 AISC Prize Bridge Awards

DECK DESIGN DATA SHEET

NO. 5

A SIMPLE REVIEW PROCEDURE FOR DETERMINING THE NEGATIVE MOMENT CAPACITY OF REINFORCED SLABS FORMED ON STEEL DECK.



t=total slab thickness, from bottom of deck to top of slab, inches.
P=pitch of deck (center to center of ribs), inches.
d_d=depth of deck, inches
d=distance to center of reinforcing steel from bottom of deck, inches.
A_s=area of reinforcing steel (not the deck), sq. inches/ft.
b_p=average width of one rib, inches.
$b=12\,(b_p)/P$, inches per ft. of width.
Conventional reinforcement concrete design procedures apply—such as the elastic method from ACI 318-63.

$f'_c=3000$ psi
$f_c=1350$ psi
$f_s=30000$ psi
$n=9$
example values

$P=A_s/bd$
$k=\sqrt{2pn+(pn)^2}-pn$
$j=1-k/3$

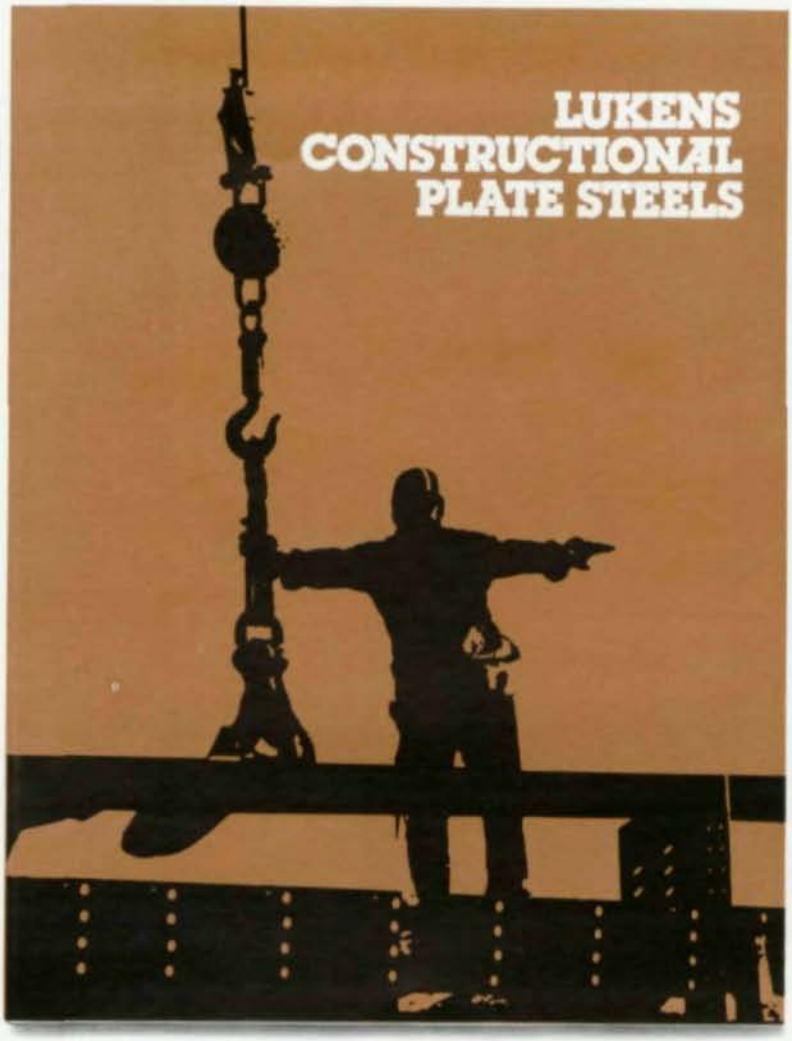
$M_c=\frac{1}{2}f_ckjbd^2$ (or) $M_s=A_sf_sjd$ (least value governs, inch pounds)

UNITED STEEL DECK, INC. PROFILE	w/h	b_p	P	b
1.5″ Lok—Floor	3.85	6	12	6
2″ Lok—Floor	3	6	12	6
3″ Lok—Floor	2	6	12	6
N—Lok	0.75	2.25	8	3.375
B—Lok	1.5	2.25	6	4.5
Inverted B—Lok	2.5	3.75	6	7.5

Notes: 1.) If the deck is unshored (during the pour) the design moment does not need to include the weight of the slab.
2.) This design procedure is applicable for either composite or non-composite deck.
3.) The w/h values are used to determine stud strength if composite beams are being used. Note that N-Lok is not efficient for composite beams.

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

00277



When it comes to constructional plate steels, we wrote the book. This current edition features Lukens' capabilities with regard to:

Sizes. Standard specification plates available in widths to 195," lengths to 1250" and thicknesses to 25." A size card shows details.

Specifications. Mechanical properties and chemistry of the various grades of steel most frequently found in bridges and buildings. Displayed in chart form.

Heat Treating. Offered on plates up to 890" long.

Stripped Plate. An alternative to universal mill plate in applications such as fabricated bridge girders. Produced in lengths from 120" to 1250," widths 12" to 48" and thicknesses ¾" to 12."

Lukens-Conshohocken. A rolling mill and shipping complex designed to meet your needs for light-to-medium thickness carbon plate and our Sure-Foot® safety floor plate.

Lukens Fineline.® A family of low-sulfur constructional steels particularly effective when used in fracture critical applications.

For your copy of this brochure, illustrated with photos of our facilities and our products in use, just fill out the coupon below.

LUKENS STEEL

Write right now.

LUKENS STEEL COMPANY
586 Services Building
Coatesville, PA 19320

Please send me a copy of your brochure, LUKENS CONSTRUCTIONAL PLATE STEELS.

NAME ______________________

TITLE ______________________

COMPANY ______________________

ADDRESS ______________________

CITY ____________ STATE ____________ ZIP ____________

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 XXV NUMBER 1/FIRST QUARTER 1985

CONTENTS

1985 INTERNATIONAL ENGINEERING SYMPOSIUM OFFERS UNIQUE OPPORTUNITY

The 1985 International Engineering Symposium on Structural Steel, jointly sponsored by the American Institute of Steel Construction and the Canadian Institute of Steel Construction, offers a unique opportunity for the international engineering community to hear some of the world's leading authorities in structural engineering research and design. The Symposium, May 22-24 at the Palmer House in Chicago, replaces the annual AISC National Engineering Conference for 1985.

The host city has Magnificent examples of steel construction, including three of the tallest buildings in the U.S. One steel-framed high-rise now under construction, Northwestern Atrium Center, will be the subject of a special panel analyzing decision making processes during framing material selection. Experts from the Far East will discuss steel's impact on that region's dynamic economic growth as one speaker focuses on high-rise construction, another on trends in cable-stayed bridges. Seismic design, wind action problems, cable structures and space frames will receive special attention. Sessions on advances in short-span steel bridge design and bridge retrofitting are scheduled. Canadian input includes developments in limit states design and research on steel plate sheer walls.

In addition to innovation and economy, quality assurance is one of the most important aspects of steel construction. Specific emphasis will include the effect of weld repairs by a representative of England's Welding Institute and quality assurance procedures by an American specialist.

For more detailed information, turn to the ad on pg. 20 of this issue.

One Financial Place: Exchanging Times in the South Loop

by Raymond S. Clark and John Zils

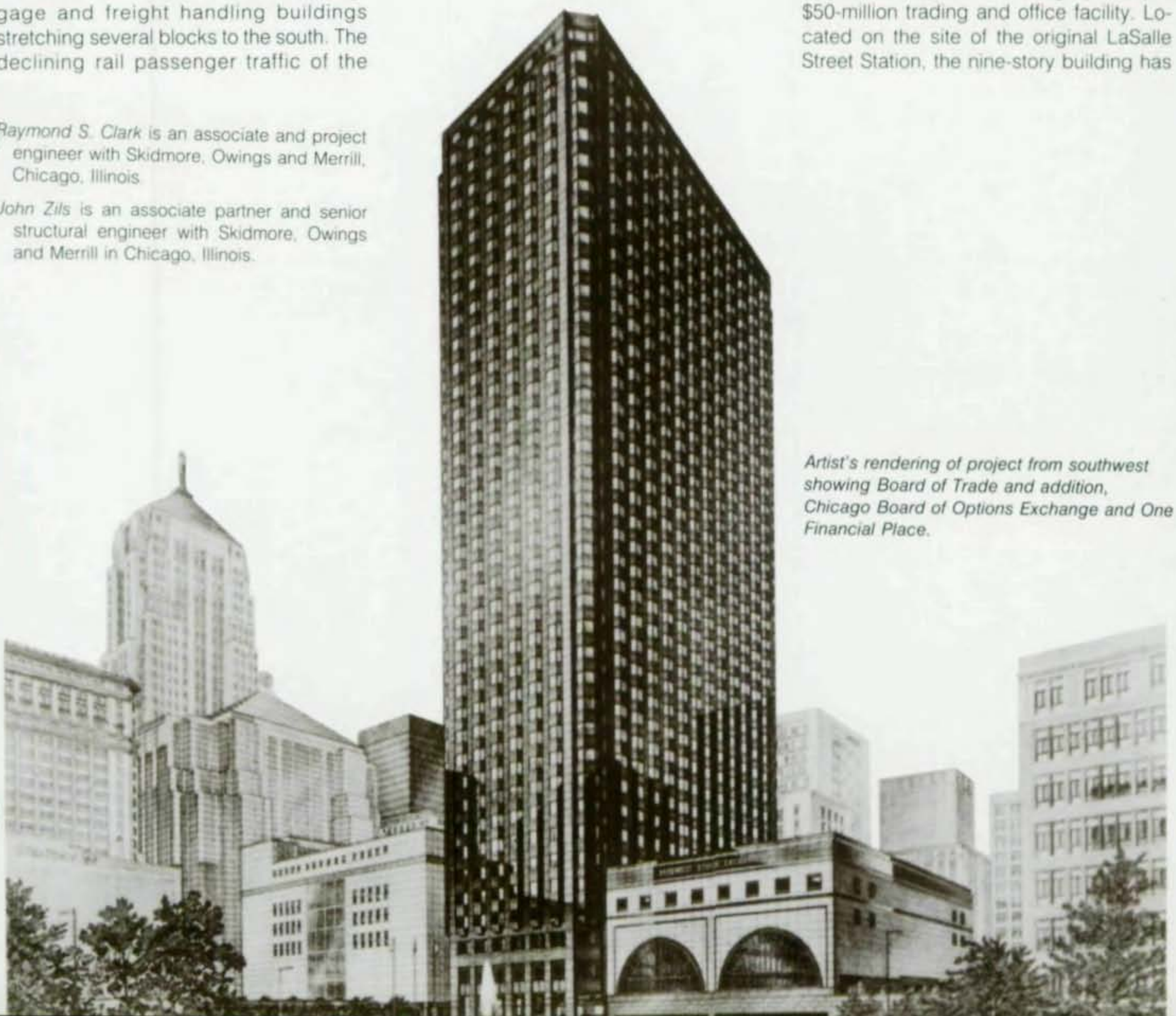
Recent developments in Chicago's south Loop are rapidly transforming what was once one of the city's major rail transportation centers of the past into an extension of the LaSalle Street financial district: an Exchange Center of the future. Since 1902, the LaSalle Street Railroad Station served as the primary metropolitan terminal handling passengers, mail and railway express for the Chicago, Rock Island and Pacific and New York Central Railroads. The 13-story station house, at Van Buren Street between LaSalle and Sherman Streets, served as the northernmost element in a series of railroad facilities, elevated track structures, and baggage and freight handling buildings stretching several blocks to the south. The declining rail passenger traffic of the 1950's and the later relocation of a consolidated Amtrak passenger service to Chicago's Union Station eliminated the need for a major rail terminal at LaSalle Street Station. As time and the forces of nature took their toll on the facilities, railroad officials decided to abandon the building. Planning began for a new, smaller passenger terminal, to be located south of the original terminal. The new terminal, intended to accommodate the computer rail traffic serving the southwest suburbs, was to be operated by the Regional Transportation Authority (RTA). The original station and baggage handling facilities from Van Buren Street to Congress Parkway were razed in 1979, and the site was cleared and prepared for new construction.

Raymond S. Clark is an associate and project engineer with Skidmore, Owings and Merrill, Chicago, Illinois.

John Zils is an associate partner and senior structural engineer with Skidmore, Owings and Merrill in Chicago, Illinois.

New Development

The first project to be developed in this Loop area was the addition to the Chicago Board of Trade Building (CBOT). This 1983 AISC Award-winning facility is located north of Van Buren Street (see Fig. 1). In 1980, construction began on the new Chicago Board Options Exchange (CBOE), a \$50-million trading and office facility. Located on the site of the original LaSalle Street Station, the nine-story building has



Artist's rendering of project from southwest showing Board of Trade and addition, Chicago Board of Options Exchange and One Financial Place.

been occupied and operational since February 1984. The latest addition to this exciting devloping corridor is One Financial Place, a 39-story, one-million sq ft office tower between LaSalle Street and Financial Place (formerly Sherman Street) and CBOE and Congress Parkway. A particularly unique feature of the One Financial Place project is the six-story, 85,000-sq ft low-rise structure located south of the tower and directly over the eight-lane Congress Parkway (Eisenhower Expressway). This facility features the new trading floor for the Midwest Stock Exchange, as well as the new RTA Commuter Station. It is sited upon the railroad bridge structure originally constructed to allow trains to pass over the expressway and into LaSalle Street Station.

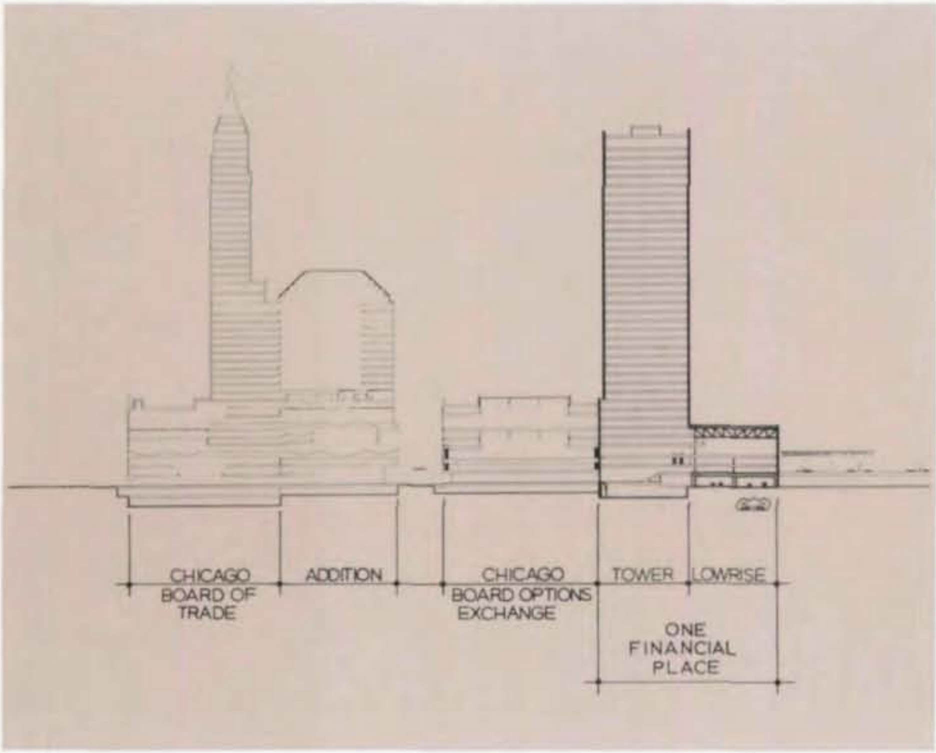
Building for the Future

Architectural programming of the One Financial Place project took into consideration numerous physical, functional and marketing aspects. It was the owner's desire to produce a first-class building responsive to the needs of the major tenant, the Midwest Stock Exchange (MSE), while providing state-of-the-art services for the general office tenant environment. MSE functional requirements included a 25,000-sq ft clear-span trading floor, extensive computer systems installation, an uninterruptable power source and special mechanical support facilities. While it was desirable to maintain a direct access between the CBOE and MSE operations, provision was necessary for adequate security for the separate exchanges, their computer facilities and vault areas. Creature comfort was an issue because of the heavy expressway traffic below the low-rise building and the almost continuous commuter train activity immediately south of the building. Sound and vibration readings were taken throughout the site to determine the acoustical/damping requirements for the new structure.

In addition to MSE program requirements, the development was intended to house new commuter station facilities. The RTA program called for a waiting room, ticket offices, restrooms and office space. The new station was required to be self-contained and separated from other building functions. Since the existing bridge is the only means of access from the south commuter platforms to the north side of the expressway, it was necessary to provide concourses through the low-rise building, into the tower and down to ground level. The peak rush-hour pedestrian traffic on the concourses is approximately 15,000 passengers.

Preliminary structural systems development began with a general knowledge of the functional requirements of the building. In an attempt to use the entire site and maximize the size of the typical floor, systems studies proceeded in reinforced concrete, structural steel and composite frame. The predominant site constraints were the adjacency of the new CBOE building to the north and the existing railroad bridge and abutment wall to the south. Both structures are built adjacent to the property line. A more important constraint than the above grade structure was the existing caisson foundations for these two adjacent structures. The foundations, in most cases, projected over the property line at bell elevation. Existing bells were of varying sizes and located at depths from -49 ft Chicago City Datum (CCD) to -60 ft. The CBOE caissons were constructed in 1980, while the bridge caissons were placed in 1946 and 1951. It was soon recognized that the size of a typical floor in the new office tower would be dependent on placement of exterior columns, somewhat predetermined by the clearances available to the neighboring caissons. Structural systems selection was also affected by the foundation bearing capacity of the hard clay strata and effects of placing new tower caissons extremely close to the CBOE and bridge caissons.

Fig. 1. Elevation looking east at recently developed office and trading facilities.



Foundation Constraints

Foundation design studies were based on an original design concept calling for a 40- to 60-story building, with perimeter frame columns spaced at 15 ft o.c. and a 40-ft wide shear core. The reinforced concrete schemes studied were found to be too massive. The heavy concrete structural dead loads limited the total number of floors that could be constructed, given the restrictive adjacency constraint. A structural steel tube-frame building with vertically braced core produced the lightest and most efficient structural system. The program requirement for 1,000,000 sq ft was attainable, while allowing for a cost effective caisson solution. Due to the adjacency limitations, four alternative solutions were generated. All four solutions were evaluated on the basis of cost, construction time and impact upon the architectural program requirements, with solution No. 4 selected as the most appropriate.

1. Rock caissons installed into bedrock at -90 ft CCD. This alternative would allow for a much heavier building, but was rejected due to the increased cost (nearly $750,000 more than hardpan caissons) and the extended foundation construction time.
2. Belled caissons at -50 ft CCD (same as CBOE) with a bearing pressure of 16 ksf. This solution would have required extremely large caisson bells for the anticipated perimeter column loads. Since the bell eleveation is the same as CBOE, clearance must be maintained between

bells, requiring that perimeter columns be moved inward from the property line, and reducing the typical floor size. This alternative was rejected as inconsistent with the basic programming objective to maximize the typical floor size.

3. Similar to 2 above; except provide the desired floor size and exterior tube grid above grade and shift the vertical framing system inward toward the core at grade level. This shift in exterior column placement away from adjacent caissons was to be accomplished using an elaborate transfer system of tension ties and grade beams. The alternative was rejected due to cost and complexity.
4. Belled caissons extended to the deep hardpan clay stratum at -70 ft CCD, at or just above an extremely dense layer of saturated silt and having a bearing capacity of 40 ksf. The solution allowed for perimeter caisson shaft placement very near the property lines and unobstructed bell placement at the deeper elevation. Special drilling procedures and permanent steel shaft liners were required when drilling close to existing caisson bells. Geotechnical analysis revealed that the new caissons, properly constructed at an elevation 10 to 20 ft below adjacent bells, would have negligible effects on the CBOE or bridge structures.

Tower Structural System

The structural steel framing system was selected for several reasons:

1. Reduction in total weight of building structure to facilitate the foundation design.
2. Speed of erection, allowing for a compressed construction time and earlier MSE occupancy.
3. Long term flexibility in accommodating special tenant requirements.

The 39-story 1,095,000-sq ft tower is comprised of a single basement, building lobby, pedestrian concourse at Level 2, 36 office floors and a private hotel at Level 39. The 120x210-ft tower has a lease span of 40 ft north and south of the core and 45 ft at the east and west ends of the typical floor. The exterior tube column grid is 15 ft o.c. with a central core spanning 40 ft. The typical floor area is approximately 25,000 sq ft.

The structural concept was to optimize lateral and gravity systems to produce the simplest, most efficient use of materials, yet satisfy the design criteria. As previously mentioned, the lateral resistance system consists of an exterior tube frame with columns spaced at 15-ft centers (see Fig. 2). The tube is complemented in the north/south direction by the five 40-ft wide vertically K-braced core trusses. The combination of tube and core bracing in a 39-story building made it possible to tune the structure, thereby maximizing the efficiency of both systems and optimizing the use of material. Wind resistance was adjusted through the variation of column-spandrel proportions and core bracing member sizes. This iterative process was accomplished using a single quadrant, unlumped computer model representative of the doubly symmetrical structural system. The final distribution of gravity and wind forces between exterior tube and core resulted in an optimum balance of stress and drift control.

Core columns consist of cover plated W14x730 shapes at the base and reduce to W14x68 at the roof penthouse. Core bracing members are double WT5 shapes. Tube columns are wide-flange shapes built-up as three plate weldments for the lower four floors, then transitioning to standard rolled shapes ranging from W36 to W27. Spandrel beams range in size from W36x300 to W27x84. All exterior columns are 50 ksi material except the uppermost seven levels of 36 ksi. Spandrel beams are 36 ksi. Due to the stress levels in the tube frame, continuity plates and web doubler plates were typically not required, again reducing fabrication costs

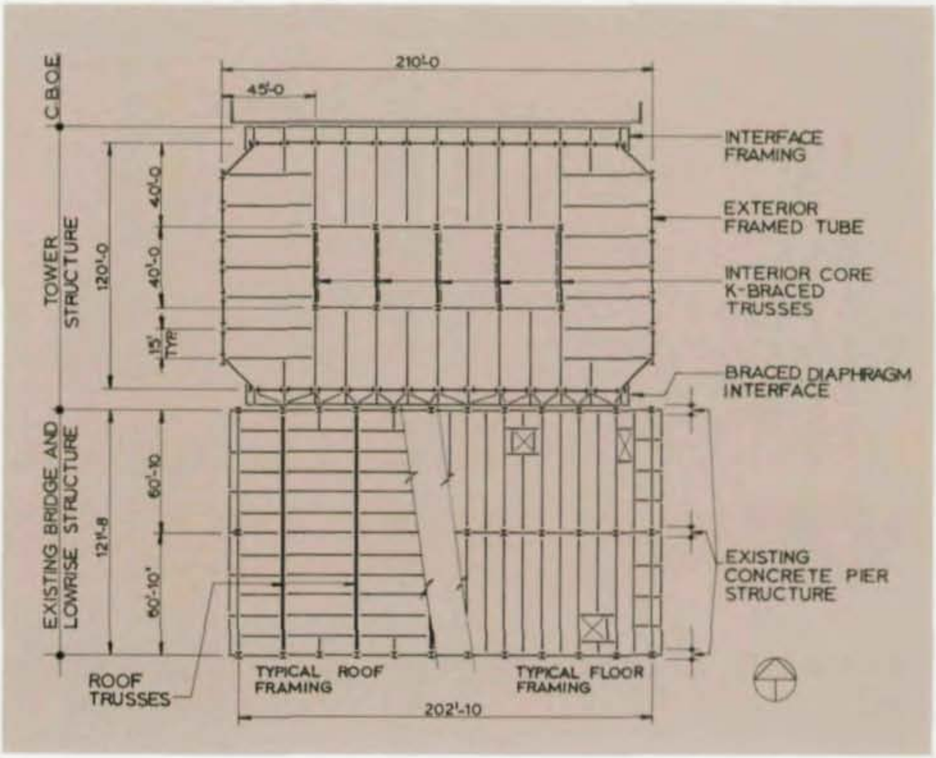
Fig. 3. Typical 45° corner spandrel condition and cantilevered floor slabs, creating bay window articulation.

and time. The typical erection unit for the perimeter frame was the standard two-tier, shop-fabricated column/spandrel tree. The column-to-spandrel moment connections were shop welded, using a combination of partial penetration and reinforcing fillet welds. Special "offset spandrel" erection units were required at the mechanical plenums (two bays, north facade), and at the 45° chamfered corners of the tube. In these locations flange continuity plates were employed (Fig. 3).

Tower Floor Framing

The typical floor framing system consists of composite beams using standard rolled shapes: W21 spanning 40 ft and W24 spanning 45 ft. All beams are spaced on the tube grid at 15 ft o.c., except where heavier loadings required 7.5-ft or 5-ft spacing. Typical office slab framing consists of 3-in. noncellular 16-ga. composite metal deck spanning 15 ft, with a 2½-in.

Fig. 2. Framing and steel structural system for both tower and low-rise buildings.



lightweight concrete topping slab. Heavier conventionally reinforced slabs were required for the lobby and public spaces, as well as some MSE floors. Specific MSE floor loading criteria called for capacities ranging from the building standard, 50 psf + 20 psf partitions, to 225 psf for special computer power support facilities. All MSE requirements were extensively researched and carefully documented prior to final framing design. In some areas, the slab thickness was increased to 3-in. deck plus 3 ¼-in. concrete, using normal weight concrete to provide for vibration damping. As part of the architectural articulation of the exterior granite facade system, bay windows were provided at each typical and each corner column bay. The facade system is supported by cantilevered floor slabs at each level (Fig. 3).

Railroad Bridge Structure

The LaSalle Street Station mailroom, railway express offices and baggage handling facilities originally extended from the station house southward to Harrison Street. These facilities were located directly below the railroad trackbed framing. This framing, built in 1902, consisted of heavy, riveted plate-girder construction supported on columns spaced approximately 15 ft o.c. north/south and 12 ft o.c. east/west. In 1939, the Chicago Department of Public Works began plans for a section of the Congress Street Expressway (later redesignated Eisenhower Expressway) which would extend from Wells Street to Clark Street. Since the eight-lane divided highway was to pass through the existing facilities and beneath the railroad trackbed framing, it was necessary to replace the 1902 structure with a new, column-free bridge structure. A condition imposed by the railroad companies was that all train operations be allowed to continue functioning uninterrupted during the phased bridge construction.

Design of a two-span pier supported structural steel bridge structure proceeded during the 1940's. The structure consists of 11 individual track bed units and six passenger platform units, each spanning approximately 60 ft-2in. between piers (see Fig. 4). Each track unit is made up of four W36 x 230 girders tied together at 7 ft-5 in. intervals by W18 riveted diaphragms and a continuous ¾-in. thick x 10-ft wide horizontal flange plate and concrete rail bed. Each platform unit consists of two W36 x 170 girders tied at a similar interval by W16 diaphragms and a continuous ⅜-in. flange plate and concrete slab. These units were preassembled offsite and transported to the site by rail for the phased erection of the bridge.

Foundation work for the bridge structure actually began in 1946 with the installation of caissons for the south and center piers. Subway construction was also proceeding at this time under the south four lanes of the expressway. The north pier caissons were completed in 1951. Continuous caisson cap girders were constructed and the reinforced concrete piers placed. Final erection of the new track units and platforms was carefully sequenced so that no more than two of the 11 tracks were out of service at any one time. The bridge construction was completed in 1956, and the expressway was opened, becoming a major thoroughfare between the Loop and the western suburbs.

Planning for the "Bridge Building"

As the design team began the architectural programming of the low-rise building, a rigorous engineering investigation of the existing bridge structure was initiated. The two activities proceeded concurrently. The results of the bridge investigation eventually dictated several of the design parameters from which the facility was planned. As previously mentioned, the two primary functions to be located in the low-rise were the new RTA commuter station and the new Midwest Stock Exchange trading floor. MSE space requirements also called for an observation gallery, employee cafeteria, kitchen facilities and office areas immediately adjacent to the trading floor. Additional space planning was required for functional amenities to be provided by the owner. These include a restaurant, kitchen and dining facilities, health club, swimming pool, exercise rooms and lounges. Mechanical systems were required which could properly service the trading floor environment, while meeting the needs of all adjoining spaces and functions. The formidable task of blending all of the complex systems into one unique structure soon became apparent.

The evaluation of existing conditions of the bridge took several phases. The objective was to determine the suitability of the bridge as the foundation support for the new low-rise. A related objective was to determine the supporting capacity of the bridge in order to establish the maximum possible size and scope of the development. Review of the plot survey and site topography enabled designers to begin establishing plan and elevation relationships between the bridge and the proposed new tower. This was essential in the organizational planning of both buildings, since all floors were intended to be aligned for full interface.

The initial phase of visual examination of the bridge involved a review of typical track unit bearing details on each concrete pair. Column grids were established based on the layout of the tower at 15 ft o.c. It was determined that spacing the low-rise columns at this common grid would introduce an array of dissimilar bearing conditions. Each column base would be unique and would require either partial removal of the track unit framing or extensive detailing of stiffeners to receive the new columns. It appeared more reasonable to organize the lowrise grid so that all columns are placed directly on the tops of the concrete piers, between the track unit and platform framing (see Fig. 4). This simplification allowed for the standardization of column base details. However, this approach did produce an irregular column spacing dictated by the original layout of bridge framing. Identifying this constraint early during design development enabled the architects to plan the interface of the two buildings appropriately.

A visual inspection and testing program

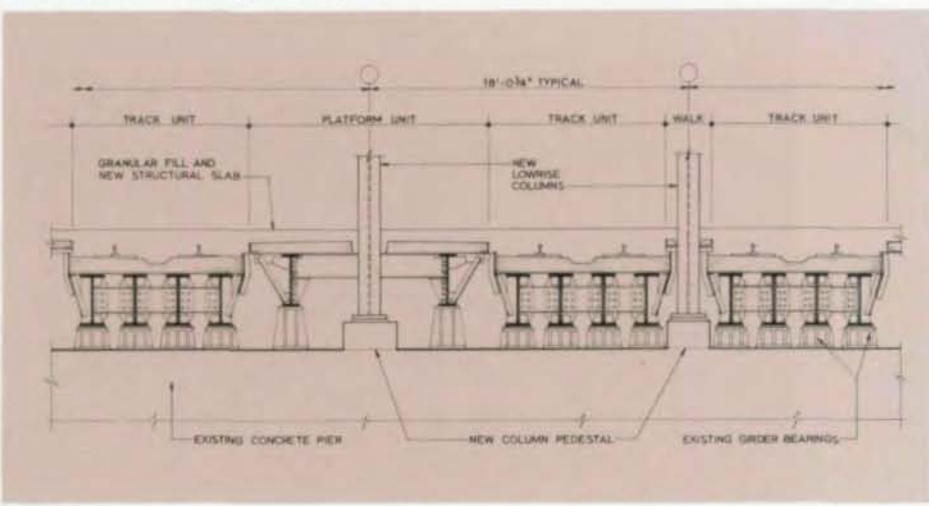


Fig. 4. Existing bridge framing of track and platform units. Note arrangement of low-rise columns to clear framing, bear on piers.

of the existing bridge was the second phase of the engineering evaluation. The general requirements for this program were specified by the structural engineer and included the following:

1. Complete visual inspection and sounding of all concrete surfaces for cracks, delamination, deterioration and corrosion. Cores were taken from all piers and cap girders to ascertain in-situ concrete strength. Chemical analyses were performed to determine the extent of chloride ion penetration.
2. Complete visual inspection of all structural steel members, connections and bearings. Extent of corrosion and loss of section was determined. The accuracy of record documents was determined by inspection and measurement of all members and connections.

Results of the inspection and testing program confirmed that the existing bridge structure was in good condition, structurally sound and that the erection of the bridge was consistent with record documents. Areas of concrete cracking and spalling were identified for repair. All structural steel framing and girder bearings were found to be properly aligned and free of cracks, fractures or warpage (see Fig. 5). Corrosion of structural steel members was found in limited areas and classified by degree. Where severe corrosion was identified, subsequent ultrasonic testing of beam flanges, cover plates and beam webs was performed to establish the extent of loss of section. Engineering analysis substantiated that although some loss approached 8% of theoretical section, the existing member load carrying capacity exceeded the new design loading criteria.

Fig. 5. View of girder bearings at north pier shows localized corrosion, areas that were cleaned and repainted.

The information provided by the inspection program was used by the structural engineer in the development of a program for repair and preventive maintenance of the bridge structure. Again, the general requirements were specified and included the following:

1. Repair by structural patching all surface distress, spalling and delaminations.
2. Cleaning and repair of all exposed and corroded reinforcing bars.
3. Pressure injection epoxy grouting of all structural concrete cracks.
4. Cleaning of all corroded structural steel and reapplication of paint.

This repair program was initiated during the tower foundation construction and was completed during the erection of the new low-rise structure.

The engineering analysis and design capacity evaluation of the bridge structure began with a thorough search for the original documents, drawings and construction logs. The structural engineer was able to obtain working drawings of the LaSalle Street Station and all facilities from the Rock Island Railroad archives. The contract drawings for the various phases of new bridge construction (1946 thru 1956) were provided by the Chicago Department of Public Works. The city was also able to provide prints of the bridge shop drawings and erection drawings, as originally prepared by American Bridge Division in 1955. Caisson construction logs for the pier foundation were available through the city. This aided greatly in the determination of existing caisson support capacity and geotechnical analysis of settlement behavior of the piers.

Floor loading criteria was developed to accommodate MSE program requirements, as well as other special functions. Due to the flexibility needed for unrestricted equipment placement on the trading floor, the reinforced concrete metal deck slabs were designed for 200 psf live load and 60 psf superimposed load. Floor beams were designed for 125 psf live load and 60 psf superimposed load and a 30,000 lb. concentrated load at any point within the span. Live loads for the other public spaces were typically 100 psf, with up to 30 psf allowed for floor finishes. The very high loading criteria had a major impact on floor beam design and final tonnage. As the column grid and floor framing was finalized, column loads and locations were determined. The evaluation of existing piers and caisson capacity verification was then completed.

Low-rise Structural System

The primary space planning requirement for the MSE trading floor called for a totally column-free space. This could only be accomplished by bearing all columns directly on the existing concrete piers, thereby resulting in a long-span structure. The conceptual development of the six-story low-rise structure led to the decision to isolate the new construction from the existing track unit/platform unit framing, due to its inability to carry the concentrated column loads and its inherent flexibility. The most logical framing system capable of meeting the span criterion while minimizing dead load was a long-span, two-bay lightweight structural steel system.

The final design layout of the low-rise structure calls for a building 121 x 208 ft. Bay spacing is approximately 18 ft east/west and the long-span condition north/south as previously noted, with each span approximately 60 ft-10 in. The tower and low-rise have a structural interface at each floor, Levels 2 through 7, consisting of a propped cantilever braced diaphragm system (Figure 2). The system actually couples the two buildings together in the north/south direction. This diaphragm provides lateral stability for the lowrise through the linkage to the tower tubular frame and core bracing. The friction-bolted connections were maintained only finger tight until the completion of the tower structural steel slabs and granite cladding erection. This allowed the linkage connection to behave as a temporary hinge and enable the tower to undergo initial caisson settlements predicted to be 0.5 in. to 0.75 in. The connections were recently torqued up to complete the lateral stability system. Lateral bracing of the low-rise in the east/west direction is provided by two bays of vertical X-bracing at the north and south facade lines. This bracing was designed to produce a displacement compatible with the tower lateral behavior.

The long-span floor framing employed wide flange shapes ranging from W30 to W36, with all beams designed for composite action. Floor slabs were either 3-in. composite metal deck and 3¼-in. lightweight slab or a 6¼-in. reinforced concrete slab on 3-in. metal deck. Certain areas of floor framing were suspended from long-span floors above in order to maintain isolation of the superstructure from the bridge framing. These areas occur at the swimming pool and racquetball spaces.

The column-free 25,000 sq ft trading floor is made possible at Level 4 by span-

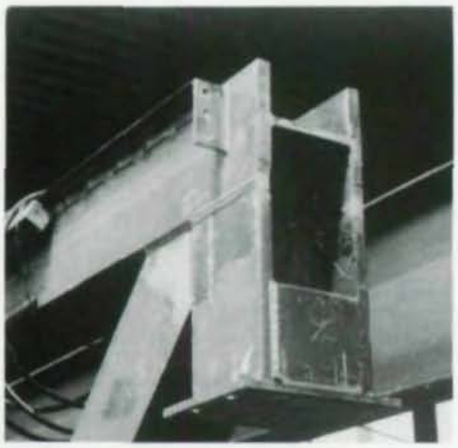


Fig. 6. Main roof trusses (above) were assembled on trading floor deck and erected by two cranes.

Fig. 7. Typical field splice details (l., above) for main truss top and bottom chords.

Fig. 8. Main truss top chord bearing (l.) simplified erection, allowed truss to be raised and placed on column cap plate.

ning the roof truss framing the full 121 ft from north pier to south pier. The trusses are fabricated using wide flange W14 shapes exclusively (see Fig. 6). The five main trusses are 10-ft deep, w.p. to w.p., and weigh approximately 55 tons each. Spacing of roof trusses is 36 ft with secondary roof framing consisting of W21 purlins at 10 ft o.c. and a composite metal deck slab. The trusses are crowned to a w.p. depth at midspan of 10 ft-7 in. This allows for the necessary roof fill drainage configuration. In addition to the truss crown, all trusses are cambered up to 3 in. to allow for dead-load deflections. Due to the size and weight of the trusses, all were fabricated in three sections. High-strength friction bolted splices were provided, allowing the trusses to be assembled in the field and lifted into place (see Fig. 7). The trusses were detailed using a "stub column" element at the ends of the top chord to facilitate erection (see Fig. 8).

Structural Steel Erection

Erection of the tower was initially handled by a Manitowoc 4100 crawler type mobile crane. As the tower structure proceeded, a self-hoisting FMC 1900 tower crane was assembled in the core and utilized for the balance of tower steel erection. Due to the straightforward structural framing concept employed and excellent logistical planning by the general contractor and steel erector, the entire 40-level tower was erected in just six months. This record breaking performance was a full three months ahead of schedule.

Erection of the low-rise was done by dividing the building horizontally into three separate divisions. Each division was erected to its full height, including placement of roof trusses and purlins, prior to starting the next division. The steel erector used a 100-ton mobile crane placed on the bridge structure and moved about on heavy timber mats.

Scheduled for occupancy in November 1984, One Financial Place stands as a shining example of successful structural planning, efficient design and effective construction procedures. Given the unique combination of site constraints, existing structures and program requirements, this project owes much of its success to the appropriate use of structural steel. □

Architect/Structural Engineer
Skidmore, Owings & Merrill
Chicago, Illinois

General Contractor
Continental-Turner Construction
Chicago, Illinois

Steel Fabricator
F.M. Weaver
Melrose Park, Illinois

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

Owner
One Financial Place Partnership, with
Casati-Heise, Managing General Partner
U.S. Equities, Development Partner
Midwest Stock Exchange, Investment Partner
Winthrop Financial, Investment Partner

clean steel by Pangborn

Engineered descaling and surface preparation systems for steel fabricators.

Now you can descale steel faster, cleaner and safer. And at the lowest possible operating and maintenance cost per ton of product cleaned.

Today, Pangborn Rotoblast® descaling is the most efficient way to clean steel products . . . plates, sheets, coils, rods, billets, slabs and rolled or fabricated structures of all types and shapes from the very large to the very small.

A Pangborn Rotoblast system can clean, prepare and provide the proper surface finish for the application of modern coating systems. Additionally, Pangborn descaling can lengthen tool and die life and improve burning, welding and machining operations.

Pangborn's horizontal roller conveyor descaler, with inverted "T" opening, handles wide flange beams up to 36" high, steel plate up to 60" wide and a variety of rolled and fabricated shapes.

Pangborn offers a full line of standard designs. Or we can custom engineer and manufacture a complete system to your special requirements . . . any size, any shape, any speed to fit into your manufacturing or fabricating process. And all of our descaling systems are built with traditional Pangborn quality, and are competitively priced.

For the most efficient, cost cutting descaling and surface preparation systems . . . talk to Pangborn first. We put over 75 years of applications engineering, manufacturing know-how and full service back-up into everything we build. Contact: Pangborn, Box 380, Hagerstown, Maryland 21740. (301) 739-3500.

Talk to the leader . . . first.

PANGBORN

Why do over 2,000 professional engineers, educators and architects belong to AISC? ...As professional members of AISC, they enjoy these benefits and privileges...<u>you can too</u>!!!

Become an AISC Professional Member NOW!

Membership in AISC directly benefits you. For a $50 annual membership fee, you receive:

- Subscription to *Engineering Journal*
- Subscription to *Modern Steel Construction*
- Regular issues of *AISC News* about events and activities of the Institute
- Bi-monthly Government Affairs newsletter about administrative and legislative action that impacts our industry.
- Up-to-date professional knowledge on structural steel's latest developments
- An opportunity to serve on special Technical Advisory Committees, and also, if invited, to become a member of a prestigious AISC Standing Committee.
- Current AISC Membership Directory
- Single copies of AISC literature and material
- A 25% discount on all AISC publications
- Complimentary copy of the 8th Edition *Manual For Steel Construction*
- Group Life Insurance information and enrollment privileges
- Immediate access to AISC's field force of 20 Regional Representatives
- An opportunity to attend AISC meetings such as the *National Engineering Conference* and the 1985 *International Engineering Symposium on Structural Steel*
- Authorization to use the AISC seal on all letterheads and promotion
- A certificate of membership and a personal membership card

So don't delay!

Return the tear-off

Form now...to

American Institute of Steel Construction, Inc.
Membership Department
400 North Michigan Ave.
Chicago, IL 60611

☐ **Please send membership application at once**

☐ **Please send additional membership information about AISC**

Name ____________________ Title ____________________

Address ____

City ____________________ State __________ Zip __________

Computer Capabilities In-House: The Leading Edge in Industry Trends

by Leonard M. Rand

Computer analysis and design has become the focal point of a major industry trend. Without repeating a long list of published market survey statistics, it is sufficient to say that the A/E/C industry is in the middle of a major rush to computerize. This move to automate includes office productivity, accounting, project management, CAD and computer-aided engineering (CAE). The real reasons the A/E/C industry is plunging headlong into the computer world are very simple, pragmatic justifications. But the manner in which many firms approach this new frontier is anything but simple and pragmatic. This article focuses on the computerization opportunity and, through case studies, offers insight and direction to firms that are involved, or soon will be, in this often intimidating process.

Pragmatic Reasons to Computerize

All of the valid reasons for a firm to computerize its operations boil down to just a few:

- Rising costs demand higher productivity from every employee.
- The industry demands more output in the same time period, or the same output in a compressed time period.
- Expensive money and squeezed cash-flow demand rigid control of finances.
- More and more clients expect their A/E, E/A, A/E/C and consulting engineering firms to be current with the industry trends—on the leading edge.
- Much of today's innovative architectural design requires a computer for structural analysis and design. An increasing number of projects could not be undertaken without a computer.
- Small service bureau users are restricted in the flexibility and cost efficiency of their computer capability.

Surveys indicate that somewhere between 50% and 75% of A/E, E/A, A/E/C and consulting engineering firms have some internal operations on a computer that the firm owns. However, the surveys also clearly show that many of the firms who have computers in-house do not use them for structural analysis and design. The overriding reason for many firms' hesitation to install CAE on an in-house computer system appears to be that engineering management has great difficulty finding structural software to fulfill their requirements.

Two Groups Rush to Internalize CAE

Even with the difficulty many firms face in finding and selecting appropriate structural software, the number of firms internalizing CAE grows at somewhere between 20% and 40% a year. Two major factions of the A/E/C industry comprise the rush to in-house CAE. Each group has its own unique set of problems and needs. Each approaches the search and purchase of CAE hardware and software differently.

The first group is made up of firms who have previous CAE experience through external resources, such as service bureaus. These firms are familiar with some form of mainframe-based structural analysis (and sometimes design) software. These firms are, as a rule, well informed on the CAE subject and can evaluate objectively hardware and software options if supplied sufficient objective information. Their evaluation of an in-house system will be placed in the framework of previous CAE experience. What these firms usually seek is a hardware-software system that is at least as versatile, sophisticated and powerful as the mainframe-based resource they have been using. They are purchasing everything from PCs to super mini-computers.

The second group, however, lacks such CAE experience. They are the small firms who are taking their first plunge into CAE and are doing so with the in-house approach. These firms are basically buying PCs and then looking for software to drive their new computers. With no performance benchmark to guide them, they can easily overestimate what an inexpensive piece of hardware and/or software can do for them. Because of budget limitations, they sometimes allow themselves to think only in terms of today's needs vs. tomorrow's requirements. These firms are sometimes tempted to spend heavily on the hardware (relative to their budget constraints) and compensate by skimping on software. Invariably, such a course results in dissatisfaction with the software that is deemed inadequate within six months.

Mixed Approach to Software Solutions

Depending on prior experience and in-house expertise, firms internalizing CAE approach their ultimate software solutions in a variety of ways. Some of the bigger firms write software programs for a portion of their CAE needs. However, the majority of the software they use is purchased from third-party developers. The home grown software programs are often the simple structural utility programs used to tackle the simple, day-to-day tasks—not the powerful analysis and design software needed for major projects. These comprehensive, integrated software systems require several man years of sophisticated development and programming effort—not feasible for these firms to undertake.

The smaller firms with no ability to write their own programs buy the software they feel they can afford. Based on current purchase patterns, many of these firms appear to be buying small software programs that are pieces of a total solution. Feeling that a total in-house solution is unaffordable, they do what their software pieces allow them to do in-house, and use a remote service bureau for the larger, more complex projects their in-house system cannot handle. Such a decision may have been based on budget constraints, or may have evolved because the firm could not find a comprehensive software system which would serve all their needs.

A third group seeks to purchase a complete CAE solution that will internalize the entire analysis and design requirements, with no need for service bureaus. These firms look for a system to integrate all their CAE needs into a single software system they will not outgrow. They want the system to be micro- or mini-computer based. Ideally, they would like software to serve their needs with equal ease whether it was running on stand alone workstations or driving a multi-terminal system. Most of these firms are still looking.

Current Perceptions of CAE Software

All of the purchases of CAE software to date have been based on the market's perception of the structural software available at the time. Some perceptions are accurate. Far

---

Leonard M. Rand, P.E. is president and founder of The Rand Group, Inc., Dallas, Texas, a major U.S. developer of leading edge, finite element-based structural analysis and design software for the architectural/engineering/construction industry.

too often, however, purchase decisions have been a consequence of misinformation and inaccurate perception. To compound the difficulty, the marketplace is a very dynamic one. What was quite true last year may be totally inaccurate today. For example, a prevalent perception is that the power, versatility and sophistication of software is directly related to the hardware on which it runs. In other words, mainframe-based software is the most powerful, versatile and sophisticated software on the market. Mini-computer software is second and *micro-based software is the least of all.* Such a perception was true a year or two ago. It is certainly not true in all cases today. Yet, too many firms make buying decisions based on such ideas.

Because of the perceived cost, far too many small firms may also be selling themselves short by assuming the only option available to a small budget is to buy a piece of the total solution now and start all over with a new software (and maybe hardware) solution later on when more money is available. New advances in micro computer-based software systems have already eliminated the need for most firms to think in terms of such planned obsolescence.

Several articles in magazines, newspapers and journals throughout the A/E/C industry have spotlighted another problem inherent to the internalized CAE trend. Too many less informed engineers with little or no previous CAE experience are purchasing inadequate analysis software and attempting to use it for structural projects outside the abilities of their in-house software. Fortunately, most of these engineers soon discover the inadequacies of their solution, and correct the problem. This realization often requires a firm to scrap the software already purchased and seek out an appropriate software tool which will do what they need it to do. This process, with its inherent false start, is usually a consequence of bad perception and inadequate knowledge of the CAE software marketplace.

Fig. 1. St. Francis Hospital. Photo shows girder layout for hexagon-shaped 11-story moment-resisting frame.

Learning from the Past

Those firms currently in the process of internalizing CAE do so because they see it as a step necessary to survival and prosperity. They are correct in their assessment of the situation. The trouble is, most firms do not have the luxury of time on their side. The demands of the marketplace no longer allow firms to take 10 years to find a suitable in-house solution. The process can be tackled rather quickly. All that has to be done is to learn from firms who have already gone through the process and have found excellent solutions for in-house CAE requirements. The following three case studies will answer many questions and give excellent direction to firms of all sizes who are currently in the midst of the march toward in-house CAE systems.

HTB, Inc., Oklahoma City, Oklahoma

HTB, Inc. is an architectural/engineering firm with offices in Oklahoma City and Tulsa, Okla.; Washington, D.C.; and San Jose, Cal. Their structural engineering department at the Oklahoma City headquarters serves the entire corporate network.

The first time HTB used a computer to analyze a structure was in 1964. One of their projects had a Vierendeel truss which supported both the roof and the floor across a 54-ft span. They had to travel 25 miles to use a university computer, where the data was keypunched onto cards, proofed and fed into the computer. After the data was analyzed, HTB still felt the need to verify the results by hand calculations.

In 1966, designs for continuous-span bridge girders were done for HTB by De Leuw Cather Company. HTB engineers would complete coding forms and mail them to Chicago, where the computer was located. In those days, air mail often took two days. By 1968, HTB was designing most of their structures on a computer owned by a local college. They furnished the service person a completed code form at 5:00 p.m. each evening. He spent the night keypunching and de-bugging so he could provide the necessary output by 8:00 a.m. the next morning.

As their volume of computer-supported design work grew, the overnight system became unsatisfactory. Time constraints and limited software availability forced HTB to make a change. In 1975, they turned to the McDonnell Douglas computer center in St. Louis, Mo. One of the first projects they designed with the McDonnell Douglas computer was the 11-story west wing for St. Francis Hospital in Tulsa, Okla. (Fig. 1). This hexagonal-shaped steel, rigid-frame building was analyzed using a three-dimensional model and the McAuto version of STRUDL software. Since HTB had no in-house terminal to access the computer, their project manager flew to St. Louis and ran the program with the assistance of a McAuto analyst. With the 3-D analysis, HTB was able to refine member sizes until a very efficient steel frame that would take wind loads from all directions was designed.

HTB linked itself to a nationwide network of computers on a time-sharing basis in 1976. Their in-house terminal was linked to the Kansas City computer center to access state-of-the-art software via telephone modem. They had some programs written internally, which were also available on the computer. A prime example of projects handled with this service bureau program is the 37-story steel-framed Mid-Continent Tower designed in 1981 (Fig. 2). HTB used the SPACE IV structural analysis and design program. The extensive time-sharing charges incurred with this project prompted HTB to invest in their own computer capability. The software they selected allowed them to do much of their design work on the mini computer-based system they purchased. However, HTB discovered they were severely limited by the number of members the software could handle at one time. Software also lacked 3-D capabilities and code checking.

In 1984 HTB purchased proprietary structural analysis and design software which takes care of all their static and dynamic analysis, accommodating both 2-D and 3-D analysis and design. Keith Hinchey, P.E., senior vice president and director of structural engineering, finds this special program solves the problems of their previous software and adds many versatile features they did not expect. "One of the system's features we find most helpful is the one which enables us to exhibit the structure on our graphics terminal while the engineer works the analysis on another terminal," explains Hinchey. "Couple this attribute with the editing capabilities and the system's advantages multiply. Our previous software required us to change the entire list of member properties just to change a single member property."

Hinchey summarizes HTB's CAE experience that has spanned two decades: "Engineering software has improved steadily since we first designed our Vierendeel truss in 1964. Now offices with only a few clients can afford a computer which can solve multi-member frames. This is only possible today because software developers understand how the computer thinks and how to use the computer to its utmost capacity. They also understand engineers and engineering and produce software that solves the problems rather than contributing to the problem."

Biar & Frost, Inc., Dallas, Texas

Biar & Frost, Inc. is a small, five-person structural engineering firm. Their use of CAE spans 11 years. In the early years, most projects were handled using traditional hand calculation methods, reserving the big projects for the expensive time-sharing method. Between 30% and 50% of their projects required some computer analysis via service bureaus.

Jack Frost, one of the firm's partners, characterizes their search for internal CAE capability in terms of commitment level. "We poked around looking at computers and software out of sheer curiosity for about 10 years," Frost explains. "We looked with genuine interest for about five years. Finally, we spent about three years doing hard looking with real intent to buy. What we were looking for was something at least as good as STRUDL, which we used through the service bureaus."

In 1981, his firm invested in a medium capacity 2-D frame program that significantly reduced their dependence on the service bureau. According to Frost, "We knew it was a temporary measure. But we saw it as step one toward our own in-house capability." Limited member handling capacity and the lack of 3-D capability helped contribute to their decision to purchase a full analysis and design system in less than one year.

Biar & Frost's story is a familiar one. A specific project and its related problems determined the timing for going totally in-house with their CAE. In early 1983, they began preliminary design work on the Sid W. Richardson Pavilion—a multi-story addition to Harris Methodist Hospital in Fort Worth, Tex. (see Fig. 3). Page Southerland Page, a Fort Worth architectural firm, had designed an eight-story structure that would grow to 20 stories as the need demands. The design posed several engineering challenges that forced Biar & Frost to go to STRUDL on the service bureau computer. The engineering challenges included: a slender, asymmetrical patient tower with a two-bay frame at one end; a full-height cantilevered, semicircular stair tower with torsionally loaded spandrels—all as an external appendage to the tower; and a double-cantilevered connecting passageway between the existing structure and the new annex, which took the form of a 47-ft long box truss passing from the third and fourth floors of the existing building to the second floor of the new annex. The use of ramps installed within the box truss made the multi-floor access possible (Fig. 4).

Midway in analysis and design on the service bureau system, the service bureau enacted a policy change that sharply curtailed support for small accounts in the Dallas area. The support person Biar & Frost had dealt with was terminated and the position permanently vacated. Log-on procedures were changed without notification and nobody was able to use the system. Nearly a week of telephone calls to the home office did not resolve the access problem. The need for computer time was becoming critical. At that point, relations with the remote service bureau were terminated. "I'm sure the big users get excellent service," confided Frost. "Like so many others, we were just too small to be handled economically by the service bureau staff. Our $15,000 to $20,000 in annual fees just was not significant to them."

Simultaneous to cessation of relations with the service bureau, Biar & Frost became aware of RAND-MICAS, a finite element-based structural analysis and design software system. After only three days evaluation, they purchased the software system and a mini-computer to run it. As Frost put it, "We finally found something much better than STRUDL we could use in-house."

With the new mini computer-based system installed, the firm was able to continue its work on the Harris Hospital project. Although the existing building was a reinforced concrete frame with brick veneer, it was not assumed concrete was appropriate for the new annex. Both steel and reinforced concrete were closely examined with the new system's design post-processors before determining that a welded, moment-resisting steel frame was the best choice for this project.

In the year and a half since they acquired their current computer and software system, many things have changed. Frost describes their current system as a "quantum leap ahead of the time-sharing arrangement." Frost says it is not necessarily a matter of doing more work, but of doing the work better and still meeting tough time schedules. Originally, expensive time-sharing costs forced them to think in terms of single-run analysis. "Now, we never do anything just one way," explains Frost. "We play with it. We examine it from many points of view."

It is now their standard procedure that, after a project is analyzed, the design is always looked at from both a structural steel and a reinforced concrete point of view. One partner takes steel while the other takes concrete. On the next project, they reverse the roles. After design work is completed by both, an internal critique session takes place. From the critique, and often after a lively exchange of opinions, the firm has a well-documented point of view to present to a client.

Their in-house CAE software tool has done something else to the way they do things. As Frost put it, "We have also become a very valuable source of information for the architect which we could never have been before. Instead of just a *problem-solver* for the architect, we are a *re-*

Fig. 2. Mid Continent Tower. HTB used SPACE IV structure analysis on 37-story steel framed-tower.

source. We now provide more creative engineering solutions than we ever could with the limitations of time-sharing." He cites a recent project where the architect with whom they worked asked a question Frost had never considered before. "Can we build an asymmetrical Vierendeel truss?" Frost's reply was, "I don't know. Let's find out." Both had their answer in a matter of minutes. It was a definite yes! The architect was able to continue his design with invaluable knowledge readily available from his consulting firm.

Frost characterizes his current CAE software capabilities as "a system that really works." He explains, "It is a current, practical tool that just happens to be state-of-the-art technology. It's a system with which we can grow. We see it as a total CAE solution for us now and for many years to come. Every firm seeking to have an in-house CAE program should expect nothing less from any system they are considering."

DeSimone, Chaplin and Associates, New York City

DeSimone, Chaplin and Associates Consulting Engineers, P.C. is a medium-sized consulting firm of 20 employees with offices in Manhattan. DC&A has used a mini computer-based CAE system since 1981. The Humana Corporation headquarters building was an excellent opportunity to put the software through its paces.

As Fig. 5 illustrates, the Humana Corporation headquarters building in Louisville, Ky. is not a simple rectangular box, high-rise building. The joint architectural project of Graves/Warnecke produced a structurally complex building with six different wind-resisting moment frames in the north-south direction and four in the east-west. A typical symmetrical rectangular building normally requires the design of two exterior frames. The architectural design of the two-four story lobby and ground floor loggia, as well as the differing setbacks from the 20th floor to the roof, eliminates any structural symmetry. Of the six north-south wind frames, only two are identical. None of the east-west frames are identical. This meant that nine separate frames needed to be designed.

The building also has other major architectural features such as the flying truss supporting the 24th-story balcony which interacts with the basic structure of it. The three-dimensional effects of this and several other features had to be evaluated in conjunction with the simpler action of the wind frames. All these features imposed many engineering challenges over and above those typical of a normal high-rise building.

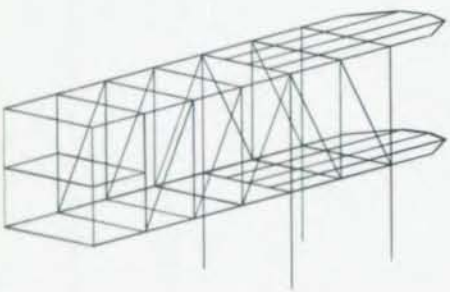
Before any serious engineering design could commence, DC&A needed to determine the structural system and material best suited for the project. Their evaluation quickly centered on reinforced concrete and structural steel. The architectural design required spans of up to 40 ft, with 9 ft clear for the floor-to-finished-ceiling. This combination of long spans and tall story-to-story heights meant that control of the building movement due to wind would be more difficult than in a normal building. Preliminary work indicated reinforced concrete member sizes would be much larger than structural steel.

The building sits on a mat foundation and the extra weight of a concrete structure also posed serious complications to the foundation design as well. A third major consideration was the flexibility of a structural steel frame to accommodate modifications for mechanical openings and beam penetrations after the structure is erected. This third consideration is not unusual, since construction of the Humana Building was to begin prior to final architectural and mechanical design, requiring a certain number of holes to be accommodated later on. All three considerations clearly pointed to structural steel as the best solution.

With preliminaries completed, DC&A began the detailed structural design. Mike Theiss, an associate, voiced the common conflict that continually challenges the consulting engineer in this phase of a project. "It is the nature of the structural design profession that some architectural concepts require greater effort on the part of the engineers than others," Theiss explains. "It is also the nature of our profession that owners do not willingly pay a compensatingly greater fee to the engineer when this occurs. Therefore, quite often, engineers are faced with making the choice between doing more work than the fee can cover or making conservative assumptions which reduce the engineers' workload but inevitably add unnecessary material and cost to the project."

Theiss finds relief for this no-win situation through his in-house CAE system. As he puts it, "The engineer's best friend in his quest to balance the work vs. fee problem is the in-house computer and a properly integrated system of software programs for analysis and design. For the one-time expenditure equivalent to a typical engineer's salary, it is finally possible to own a system that can perform a comprehensive analysis and aid in the design and checking of very large structures." DC&A now uses a finite element-based structural analysis and design system built around a relational database.

Theiss warns the engineering firm investigating CAE systems to know what to look for. He points out that not all analysis and design software is integrated. "True integration is the ability to go from analysis to design without having to manually enter results from the analytical phase, or to be able to have member sizes adjusted from the calculations of the design program to reanalyze a given problem." Theiss warns, "Most structural software packages for



Figs. 3 & 4. Architectural model of Harris Hospital Methodist, with new Richardson Pavilion in left foreground. Typical Plot (l.) shows analytical model of cantilevered box truss passageway between old and new. Plot is used as verification and documentation tool.

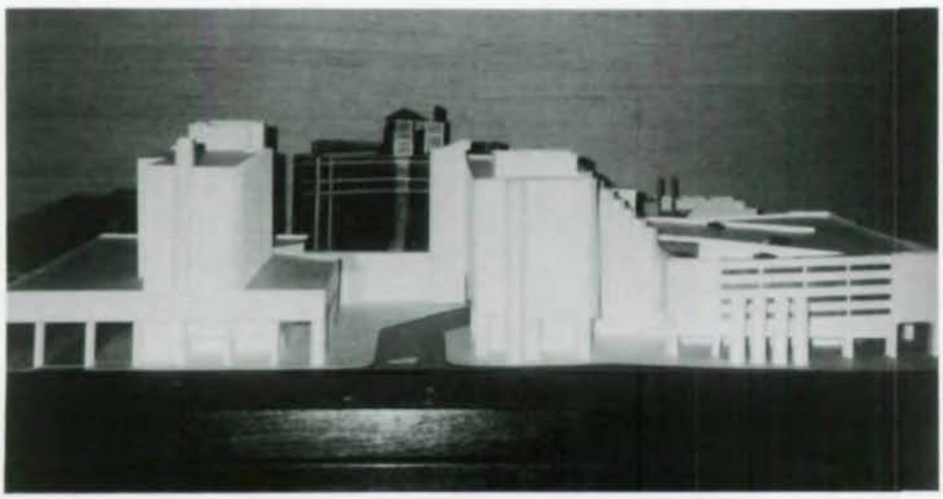


Fig. 5. Humana Corp. headquarters. Structurally complex building required design of nine separate frames.

analysis and design are not fully integrated and therefore waste valuable engineering time."

Theiss uses the Humana building as a perfect example of using the right software system to solve a most difficult problem. He explains the unusual structural nature of the Humana project would have posed a serious problem to DC&A had they not had the proper in-house CAE capability. Because of the number of separate frames and the complicated detailing on the upper stories, numerous trial runs were required to converge on an economical structural solution. As he explains, "The cost of using an outside service bureau would have posed a very real restraint on the number of trial solutions we could have investigated."

Theiss expanded on DC&A's experiences, contrasting service bureaus with in-house capabilities. "Our prior experience with service bureaus proves that access for the smaller user is not always speedy. Most engineers are minimum users compared to the large corporate users. Many of us try to use these expensive systems at night when the rates are lower. It becomes difficult to do more than one run a day. With our in-house system, we make many runs a day if necessary. With a multi-user mini computer system like ours, the engineering firm can also run more than one problem at the same time. This capability has significantly increased DC&A's ability to produce high quality engineering solutions quickly and efficiently."

DeSimone, Chaplin and Associates' experience with in-house CAE has been a very successful one. Theiss says their system has actually enhanced their ability to handle specialized, non-standard building structures—like the Humana Building. "We can handle these non-standard structures without penalizing the owner with an overly conservative design, while not penalizing DC&A by exceeding the fee," he confides. "Our computer system allows us to satisfy the owner's needs, the architect's design and our fee schedule—and we can do it quickly."

Those Who Have Done it Offer Advice

Hinchey, Frost and Theiss all offer sound advice to firms of any size who are investigating an internal CAE capability. Many of the hints listed below were voiced by all three men. All agree the process can be an intimidating one—but does not have to be. *Here is a summary of their advice:*

Seven Tips to Top Capability

1. *Understand Why You are Doing it. Keep* your true objectives clearly in mind. You want to solve as many problems and provide as many necessary tools as you can. Do not allow yourself to be blind-sided by a machine's whistles and bells—and forget that a computer does not do anything until it has software to drive it.
2. *Start with Software, Then Find Best* Computer. The most popular computer with all kinds of machine capabilities cannot compensate for inadequate software. The best software on the market does not usually need a specific hardware feature found on only one system to do its best for the engineer.
3. *For Micros and Some Minis, be Pre*pared to Spend as Much or More for Software as for Hardware. Do not expect to spend a few thousand dollars for the machine and a few hundred for the software—a sure route to short-changing yourself.
4. *Plan for Future Needs. Don't Stop at* Solving Today's Needs. Many engineers are under the impression that everyone who has a modest budget must buy a partial solution system in the beginning, then throw it out later when needs demand and budgets permit. This is not necessarily so. It is possible to find a *complete* solution which can be viewed as entry level. Study the available options before locking into planned obsolescence.
5. *Investigate Thoroughly, but Don't Make* it a Career. It is most important to make a thorough study of at least five of the best options available before choosing software and hardware. Too many engineers take years to do what a few months' study would accomplish better. Set a timetable and work hard to stick to it. Better decisions are made by weighing options fresh in memory.
6. *Buying a CAE System is Like Marriage* . . . it is a Family Affair. Even the best software around will not serve well if it does not have excellent support from the company who developed it. This support should include a strong commitment to ongoing software refinement and development. It should also include providing enhancements and new releases; providing comprehensive hands-on training for staff; providing hot-line access to professional engineers who act as first-line technical support for your questions and problems. Expect to pay for such support—at least 10% of the software purchase cost annually. *If a vendor offers free or very* cheap support, it will be worth every penny of it.
7. *Know What the Software System Ought* to Have. Then Ask Pointed Questions on Just What Their Systems Offer. Hinchey, Frost and Theiss agree you can have the best system, but only if you know what the best system ought to be able to do. They point out the best structural analysis and design system ought to have all these features as minimum requirements:
   - Be available on a variety of computers—ideally on both micros and minis. The micro version needs to be functionally identical to the mini version.
   - Do both frame- and finite-element analysis
   - Provide both 2-D *and* 3-D analysis and design
   - Perform both static and dynamic analysis
   - Be built around a true relational database so the system is integrated and highly flexible, allowing the engineer to flow from analysis to design and back again at will
   - Provide a variety of analysis postprocessors to expedite the analysis process
   - Provide resident code checking for both steel and concrete design
   - Be able to handle virtually any size structure with ease. □

Stoneman Building: (Chapter 2)
The Symphony of Flexibility Continues

by Elliot Paul Rothman and Terry A. Louderback

The Stoneman Building at Boston's Beth Israel Hospital, a Harvard teaching hospital, is a single-corridor and double-loaded floor, eight-story patient care building built in the 1950's. Its patient rooms are too small for care of acute patients in the 1980's. The hospital's program was to totally renovate Floor 8 (formerly occupied by the Labor and Delivery Unit, which moved to the newly designed Reisman Building's Floor 10) and to partially renovate patient Floors 4 through 7. As part of the $53-million project expansion, the architect planned the Stoneman Building's renovated rooms to resemble the extraordinarily successful rooms of the Reisman Building, which they also designed.

The Problem

Because patient rooms are too small, and the number of patients cannot be reduced to increase the number of single rooms or the room size, a trustee suggested, "why can't you hang the toilet rooms from the outside?" The architect and structural engineer responded with a scheme which hung a new steel structure from the east side of the building and added 1,100 sq ft per floor to five floors. The proposal worked. All double patient rooms are located on the east side of the corridor, with toilet rooms and sitting alcoves located in the new floor area; and all single rooms are located on the west side in space previously assigned to inadequately sized double rooms.

The Resolution Process—Structural Considerations

The structural engineer was consulted to find the best solution for lateral expansion of Floors 4 through 8 on the east side of the reinforced concrete Stoneman Building structure. Their immediate solution was to suspend the new space from the roof. The initial concept was to simply hang all new levels from large, cantilevered steel girders spanning from the interior corridor columns over the exterior columns. A more in-depth examination found this concept had a number of drawbacks. Interior and exterior columns do not always align in the existing structure, requiring a secondary network of transfer girders along the corridor columns. Secondly, to hang all levels would require reliance on tension hangers in the new exterior wall without a practical redundant support system.

Loads attributable to descending floor levels would further complicate deflection control, impacting upon the detailing of non-structural elements such as the curtain wall and interior partitioning. Thirdly, because the new roof-top steel would have been exposed, thermal stresses and deformation resulting from uneven heating would have to be accounted for in the design. Most importantly, suspending the entire new structure from roof cantilevers would introduce a load transfer path which the existing building could not accommodate safely. The capacity of each existing column along the east facade was computed and compared to the existing column loads, plus the net increase in column loads due to the new expansion. It was found that the upper story columns had inadequate reserve strength to safely support the loads which would result from carrying the new loads up the new exterior wall and back down the existing exterior wall. A new solution had to be found.

Ideally, the solution to the Stoneman program would address each of the drawbacks of the suspended scheme, and at the same time satisfy the architectural con-

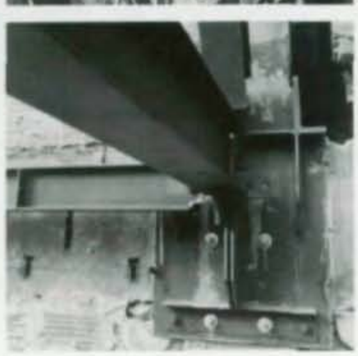


Fig. 1. Overview of new structure (above) with steel framing partially erected. Support collars at existing columns can be seen at upper floors. Fig. 4. Overview of framed brackets (above, r.) and tube column on left which provides interdependency between each bracket. Fig. 5. Support assembly with partial framing (r.) shows bolting of faceplate into existing column.

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, True and Partners, Watertown, Massachusetts.

The first "chapter" on the Reisman Building appeared in the 1st quarter, 1984 issue.

cept which required lateral expansion without supports below the fourth floor. Employing cantilevered steel girders (double channels) at each level was explored, but rejected because of unacceptably reduced headroom within the existing building. And the girders complicated relocation of numerous functioning mechanical, electrical and plumbing services; and the interior and exterior columns did not align.

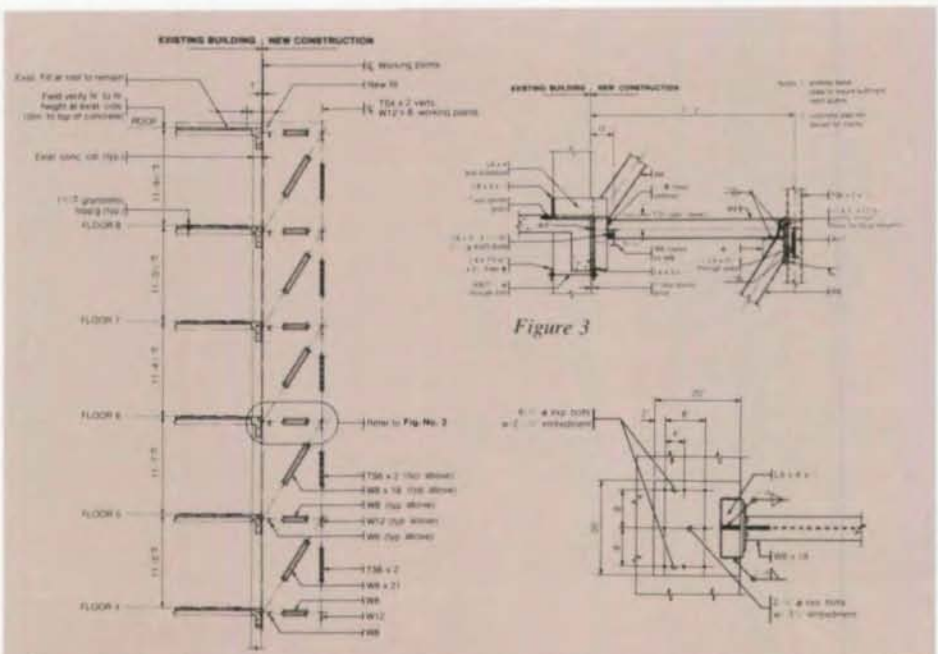
The Structural Solution

The solution overcomes the drawbacks of previous schemes. A story-high trussed "bracket" is provided at each level and column to support individual levels independently. Furthermore, all trussed "brackets" are interconnected over the five-story column height to permit load sharing and deflection control among all brackets (Figs. 1 & 2).

The transfer of new loads into existing columns presented a detailing challenge. Except at the roof and lowest level where different details were required, the horizontal component of the compression force in the diagonal struts tended to negate the tension force in the horizontal members. This resulted in the vertical force component being the primary design consideration at Levels 5, 6, 7 and 8. The resulting connection detail shown in the Figs. 3, 4 and 5 was developed. Collars convert the vertical and horizontal reaction components at each support point to bearing against the existing concrete and a couple into the existing column. Bearing against the existing concrete occurs at the exterior face of each existing column from the stiffened vertical plate as well as on the existing slab/spandrel beam adjacent to each side of each existing column from the thick 8-in. x 4-in. structural steel angles. The couple is developed by the 8-in. x 4-in. structural steel angle behind each column and through-bolted 4-in. x 3-in. angle with a 4-in. channel at the bottom of each collar. A careful analysis determined the stresses and deformations of the structural steel elements under various combinations for dead, live and lateral (wind and seismic) loads. Plate thicknesses, stiffeners and welds are designed to permit optimum behavior of the support collars.

Connection details at the roof and lowest floor must resist significant net horizontal tension and compression forces in addition to vertical forces. Different details are required at these two locations. Figure 6 shows the solution at the roofline. The horizontal tension force is resisted directly by expansion bolts in the roof and through a bolted restraint at the base of the face plate. Stiffener plates provided on the vertical face plate transfer the horizontal force for the W8 strut to the expansion and through-bolts. The relatively small vertical force component is resisted by bearing directly on the roof slab/sprandrel beam. At the lowest level, where the net horizontal force component results in compression on the existing column face, the detail used at typical floors is adapted. By using a thicker face plate and bearing angles, larger welds, and eliminating the lower restraint angle, the typical detail is made suitable at the lowest level.

To provide a reasonable degree of safety in the structure, a secondary system



Figs. 2 & 3 detail elevation at each existing column. *Fig. 6 indicates solution at roof.*

of expansion bolts between face plates and the existing concrete structure was devised. As with the design of the structural steel elements, careful analysis determined the level of shear and tension in each bolt. The size and required embedment length of each bolt is specified accordingly.

With discrete connection points at each existing exterior column, minimal demolition work is required at the existing exterior wall. The erection sequence calls for all the column collars to be installed prior to erection of trussed brackets and floor beams. There is minimal disruption to the existing structure with this concept, since all of the major steel elements are externally applied to the building. After the structural steel was installed and the new space in place, the old exterior wall was removed and the desired enlargement of the existing floor space created.

As previously mentioned, one of the chief advantages of the trussed scheme over the roof cantilever scheme is that the load transfer path is more direct. New floor and roof loads are channeled into the existing columns at their corresponding levels (or lower) so that the existing columns resist decreasing loads as they ascend. Each column and footing in the existing structure affected by the new construction has adequate reserve strength to safely support the new loads. Structural steel construction was the only solution—since light weight, durability, erectability and economy were essentials for this project.

Conclusion

The Stoneman Building includes medical/surgical and obstetrical nursing units, diagnostic cardiology, clinical neurophysiology and dialysis. Total cost for these renovations and new construction is $13,635,000. Bedrooms designed to resemble the new ones in the Reisman Building will be generated by Stoneman's lateral expansion. The lateral expansion becomes the locus for the new patient toilet rooms, a lounge/sitting area for visitors and a generous expansive window opening affording a view over Boston's Fenway Park. Patients in each room will face oak-panelled walls with integrated wardrobes, balanced by strongly organized color solutions for repose. The room is designed to increase the patient's feeling of well-being at Boston's Beth Israel Hospital—the first hospital to create a Patient Bill of Rights.

As a result of the combined new addition and total renovation, the Stoneman patient care units are brought up to the standards of the new Reisman Building, and will provide patients the same intimate quality and interior environment. The solution also extends the life span of the existing building into one equal to that of the building recently constructed. The architect and structural engineering team have once again shown that structural steel construction provided the best possible solution to a complicated expansion problem at a large medical center. □

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

Structural Engineer
Souza, True and Partners
Watertown, Massachusetts

Construction Manager
Jackson Construction Company
Dedham, Massachusetts

Steel Erector
Daniel Marr & Son Company
Boston, Massachusetts

Owner
Beth Israel Hospital
Boston, Massachusetts

Rockingham Park:
A Fast Track for a Fast Track!

by Rocco A. Blasi

Rocco A. Blasi, P.E., is chief structural engineer of Bayside Associates, a Boston, Massachusetts architectural/engineering firm.

On July 29, 1980, a disastrous fire at Rockingham Park in Salem, New Hampshire, destroyed the entire grandstand and paddock and a part of the existing clubhouse. Subsequently, the track remained vacant until July 1983, when Rockingham Venture—headed by Max Hugel, Joseph E. Carney, Jr., Dr. Thomas F. Carney and Edward J. Keelan—purchased the property. Their intent was to open the track to thoroughbred racing in May 1984.

The fast-track design and construction of Rockingham Park was accomplished with the help of architects, engineers, contractors, tradespersons and, most importantly, the officials of the Town of Salem, N.H. Their help and timely approvals were essential to complete the project on schedule.

The time element was the most critical item of the entire job, with work scheduled to start in October 1983 and the track to open on May 28, 1984. It actually opened two days early, less than eight months from groundbreaking. Due to the time constraints for this project, structural steel framing was chosen by both the consulting architect and the architect because of the speed and ease of construction with steel.

Foundation plans were issued by the architect on Sept. 19, 1983, and construction began Oct. 3. The column base plates and column sizes and lengths were issued to the fabricator in October so that an early date could be set for the mill rolling of the heavy steel columns and beams. The steel shop drawings were reviewed and checked by the engineer, usually within a 36-hour time frame. Over 100 shop drawings, including all field revisions, were approved in less than four months. Field erection requirements for a job of this magnitude were extremely accurate.

Steel erection, started in mid-December 1983, was completed in mid-March 1984. Over 1,000 tons of structural steel and miscellaneous iron were erected in less than three months.

A project of this magnitude would normally require six to eight months of design time, and one and one-half to two years for construction. The total elapsed time of ten months from design to occupancy must be a record for the industry, especially during the winter construction season in northern New England.

Space requirements were set by the consulting architect. Columns were spaced 30 ft o.c., with a stepped section 44 ft long. No interior diagonal bracing was allowed due to the open space requirements. Column-to-beam connections were made in accordance with the AISC Type 2 construction with flexible wind connection. Load-indicator bolts were used for both the gravity load shear connections and the wind-moment connections. All columns were shipped in one piece. The largest columns were those at the photo booth tower, which were W14x283, 40-ft long. The floor beams were spaced at five ft o.c. supporting a composite metal deck and a 5-in. concrete slab. The largest beams were W24x68 framing into W33x130 beams. The sloping W24x68 beams framed into W33x130 beams at one end and into W33x108 beams at the track side. Pieces cut from W18 beams were welded to the top flanges of the sloping beams to provide level platforms 7-ft wide for tables and box seats.

The intermediate beams to beams were detailed with shear connections and the beams to columns with moment connections. The sloping beams were designed as an integral part of the wind-resistant rigid frame. Only one beam required a shop modification. The entire building steel framing was completed with only three field modifications.

The roof structure cantilevers 14 ft from the main roof columns toward the track side. Triangular trusses 6-in. deep at the parapet, 12-ft deep at columns and spanning 13 ft, were designed to support the roof and its snow loads, the coping, track lighting and part of the glass wall.

The trusses were spaced 30 ft o.c. and

Completed fast track, Rockingham Park, Salem, N.H. Photo courtesy Eagle Tribune. Topping out ceremonies included (l. to r.) Paul LaRosa, Ernie Crowley, Ed Keeland, Max Hugel, Joseph Carney and Jerry Crowley.

interconnected by roof and ceiling joists. Horizontal angle frames transmitted the horizontal wind loads back to the columns. Shipped from the fabricator in one piece, the trusses connected with high-strength bolts through the truss split tees to the flanges of the main columns. They were all erected in less than two days.

The three-story judge, photo and announcer booths are over 60 ft above the ground, with the cupola over the photo booth over 75 ft above ground. This 3-story section was supported by saddle beams at the second-floor ceiling level. To visualize the photo booth, imagine a three-story plus attic home, with the first floor set 30 ft above ground. Two main beams, spaced 27 ft apart to support the photo booth, are W24x68 beams 53-ft long, including an 8½-ft cantilever. The main beams frame into column stubs at one end and are fastened to the top of a W24x68 at the cantilever end. Additional floor beams were underslung from the top of the bottom flange of the main beams to provide support to the first floor beams of the photo booth.

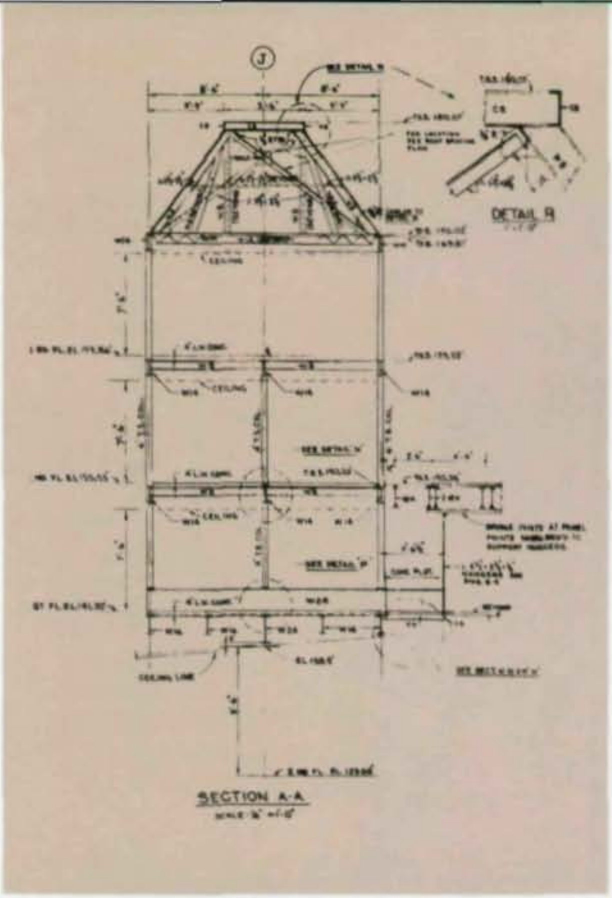
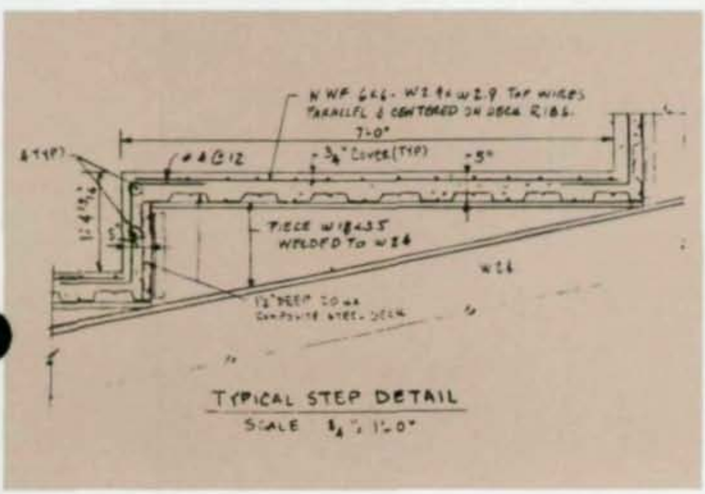


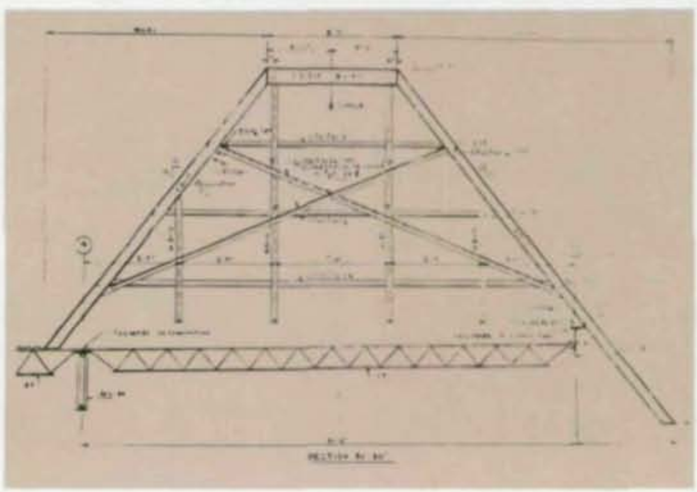
Photo booth framing is combination of steel and light steel wall studs. Detail on left.

And coming into the home stretch . . . an uninterrupted view of race from photo booth high above stadium.

Stepped grandstand (l.) shows steel framing, with typical step detail (below, l.).

Four large cupolas (r.), designed in steel for ease of construction, grace entrance canopy. Steel framing details diagramed below.





A 70-ft long enclosed passageway supported from the roof joists provides access to the first floor of the photo booth. The second floor of the photo booth has an access door to the grandstand roof.

Six columns supporting the three-story photo booth are spaced to allow an uninterrupted trackside view of the races. The floor beams, steel decking and concrete slab provide a horizontal diaphragm to transmit horizontal loads. The steel stud infill walls are braced to transmit these loads to the horizontal W24x68 beams and into the moment-resistant main building frame. The photo booth columns were designed to support vertical loads only. The entire structure had a noticeable horizontal movement until the steel wall studs and light gauge plate bracing were installed. Once the structural steel and the light-gauge steel were interconnected, there was no noticeable movement.

Large cupolas shaped like truncated pyramids were designed for the entrance canopy (4 each), the terrace canopy (3 each) at each corner of the entrance side of the grandstand and on top of the photo booth. The cupolas were designed in steel because of its non-combustible quality, its ease of construction—and the steel proved more economical than wood.

The shape of the cupolas required extensive detailing. The steel was fabricated on site at ground level and hoisted in position. Fire-resistant wood blocking and plywood decking were then attached to the steel frame. The erection procedure saved an estimated six weeks of construction time for the 10 cupolas. The largest cupolas were 30-ft square at the base and 8-ft square at the top to form a truncated pyramid of steel almost 22-ft high.

The uniqueness of this structure is not in its design, which is straightforward, but in the use of steel to achieve a remarkable speed and ease of construction. □

Architect/Engineer
Bayside Associates
Boston, Massachusetts

Consulting Architect
Bird, Fujimoto & Fish
San Diego, California

General Contractor
Crowley Corporation
Leominster, Massachusetts

Steel Fabricator
Bennington Iron Works
Bennington, Vermont

Owners
Rockingham Venture
Salem, New Hampshire

1984 AISC Prize Bridge Awards

The 1984 Prize Bridge Awards have been announced by AISC. This year 25 structures were chosen as the most handsome and functional steel bridges opened to traffic between Jan. 1, 1983 and June 30, 1984. Ten Prize Bridge Awards and five Special Awards were selected in 10 classifications (see box). Ten Award of Merit winners were also chosen for recognition.

Awards were presented to the designers of the winning and special structures at the prestigious Fourth Annual Awards Banquet in Chicago on Dec. 4. Leaders of the construction industry gathered to recognize the designers responsible for the winning bridges. Plaques adapted from the Joe Kinkel-designed sculpture, "The Long Reach," were presented to the winners by John H. Busch, AISC chairman of the board.

The Prize Bridge Competition, conducted since 1928, has over the years inspired greater attention to the aesthetics of bridge designing as well as advancing steel as a structural material.

And the winners are—

AWARD CATEGORIES

MOVABLE SPAN
Bridges with a movable span

LONG SPAN
Bridges with one or more spans over 400 ft

RECONSTRUCTED
Bridges which have undergone major rebuilding/reconstruction using structural steel to upgrade them to current traffic requirements

GRADE SEPARATION
Bridges whose basic purpose is grade separation

SHORT SPAN
Bridges with no single span 125 ft or more long

SPECIAL PURPOSE
Includes pedestrian, pipeline, airplane and other special purpose bridges not otherwise identified

RAILROAD
Bridges (non-movable) used primarily to carry a railroad, but may also be a combination railroad-highway bridge

MEDIUM SPAN, LOW CLEARANCE
Bridges with vertical clearances of less than 35 ft, with longest span not more than 400 ft nor less than 125 ft long

MEDIUM SPAN, HIGH CLEARANCE
Bridges with vertical clearances of 35 ft or more, with longest span not more than 400 ft nor less than 125 ft long

ELEVATED HIGHWAYS/VIADUCTS
Bridges with more than five spans which cross one or more established traffic lanes, and may afford access for pedestrians or parking

THE JURY OF AWARDS

STANLEY N. ALLAN
President, Harry Weese & Associates
Chicago, Illinois

EDWARD V. HOURIGAN
Director, Structures Design,
New York Dept. of Transportation
Albany, New York

RICHARD W. KARN
President, Bissell & Karn, and
President, American
Society of Civil Engineers
San Leandro, California

THOMAS R. KUESEL
Chairman, Parsons Brinckerhoff
Quade & Douglas
New York, New York

CHARLES SEIM
Principal, T.Y. Lin International
San Francisco, California

L to r: Stanley N. Allan, Charles Seim, Thomas R. Kuesel, Edward V. Hourigan and Richard W. Karn.

"The Long Reach"—a single-edition bronze by Sculptor Joe Kinkel. Winners receive a bas relief of the original sculpture which is displayed at AISC's Chicago headquarters, along with engraved names of AAE and Prize Bridge Award winners.

PRIZE BRIDGE 1984—
MOVABLE SPAN

COLUMBUS DRIVE BASCULE BRIDGE 1

Chicago, Illinois

One of the world's largest movable bridges, it carries seven lanes of traffic and two 10-ft sidewalks over a 270-ft distance between pivots. Each 6.3-million lb. leaf is supported by two A588 steel box girders, the product of innovative construction techniques, that creates a new look in bascule bridges.

Designer
Envirodyne Engineers, Inc.
Chicago, Illinois

General Contractor
Paschen Contractors, Inc.
Chicago, Illinois

Steel Fabricator
USS Fabrication Div., U.S. Steel Corp.
Orange, Texas

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

Owner
City of Chicago, Illinois

PRIZE BRIDGE 1984—
LONG-SPAN

I-470 BRIDGE over OHIO RIVER 2

Ohio County, West Virginia

Single-span, tied-arch bridge, spanning 780 ft, was acclaimed for both its beauty and economy. Box girders of A588 steel provide its stiffness and the A36 boxed rib bracing was chosen for both its architectural expression and economy. Though massive with its tapered arch ribs rising 130 ft above the girders, jurors pointed to its clean lines and excellent proportions.

Designer
Richardson, Gordon & Associates
Pittsburgh, Pennsylvania

Consultant
Deeter Ritchey Sipple, Pittsburgh

General Contractor/ Fabricator/Erector
Bristol Steel & Iron Works, Inc.
Bristol, Virginia

Owner
West Virginia DOT
Charleston, West Virginia

PRIZE BRIDGE 1984—
RECONSTRUCTED

NEWBURGH-BEACON BRIDGE NO. 1 3

Newburgh, New York

High traffic density required widening and strengthening this Hudson River bridge, which carries westbound I-84 lanes. Five hundred trusses and girders were reinforced in the 14-span, 7,855-ft bridge and it was expanded from two to three lanes with replacement stringers. The new A588 high-strength steel was painted to complement the weathering steel on the new three-span parallel bridge under construction.

Designer
Modjeski and Masters, Consulting Engineers
Harrisburg, Pennsylvania

General Contractor/Erector
American Bridge Div., U.S. Steel Corp.
Pittsburgh, Pennsylvania

Steel Fabricator
USS Fabrication Div., U.S. Steel Corp.
Orange, Texas

Owner
New York State Bridge Authority
Poughkeepsie, New York

1

3

2

PRIZE BRIDGE 1984—
GRADE SEPARATION

GRANBY ROAD over STATE RT. 137 4

Kingsport, Tennessee

Designer eliminated a medium pier and provided a 30-ft setback from the lower roadway by employing the long-span capabilities of structural steel. The 339-ft long haunched steel box girders are attached uniquely—integrally at one abutment, expansion-connected at the other—to counteract uplift forces. Jurors deemed the structure as a well-proportioned arch that appears to float over the highway.

Designer
Division of Structures, Tennessee DOT
Nashville, Tennessee

General Contractor
Phillips and Jordan, Inc.
Knoxville, Tennessee

Steel Fabricator
Carolina Steel Corporation
Greensboro, North Carolina

Owner
Tennessee Department of Transportation
Nashville, Tennessee

PRIZE BRIDGE 1984—
SHORT SPAN

SNOQUALMIE RIVER ROAD: BRIDGE 416 5

Snohomish County, Washington

Multi-curved steel superstructure provided cost-effective and aesthetic solution for replacing a timber bridge. Overhanging railings and recessed girders produced a graceful quality in the 210-ft radius curves. Built without disrupting traffic, the three-span, 272-ft bridge adds both form and function to its surroundings.

Designer
Henningson, Durham & Richardson, P.S.
Seattle, Washington

General Contractor
Dale M. Madden Construction, Inc.
Bellevue, Washington

Steel Fabricator/Erector
Fought & Company, Inc.
Tigard, Oregon

Owner
Snohomish County
Everett, Washington

PRIZE BRIDGE 1984—
SPECIAL PURPOSE

SHEEPS CROSSING TRAIL BRIDGE 6

Madera, California

A hikers' bridge, part of the vast network in the Sierra Mountains, crosses the San Joaquin in a sensitive environmental area. So not to violate the scenic area, the designer created a steel-stayed suspension bridge that could be shop-fabricated and helicoptered to the remote site. The steel was hot-dipped to minimize maintenance. A lightweight bridge with a timber deck, it adds excitement for park visitors.

Designer
U.S.D.A. Forest Service, Region 5
San Francisco, California

General Contractor/ Steel Fabricator/Erector
E. F. Owens Company
Somerset, California

Owner
U.S.D.A. Forest Service
Fresno, California

4

5

6

PRIZE BRIDGE 1984—
RAILROAD

SEABOARD COAST LINE RR. over SAVANNAH RIVER 7

Elberton, Georgia

This relocated railroad bridge, part of the R. B. Russell Lake Project, will have a final clearance of only 20 ft above water level. The massive 136-in. deep welded plate girders that span 150 ft are lost in the panorama of beauty. Steel, mostly A588, was shop-fabricated, with 57-ton girders shop-assembled, match-marked and delivered. Jurors commended the well-proportioned structure's maintenance rail to emphasize thinness.

Designer
Prybylowski and Gravino, Inc.
Atlanta, Georgia

General Contractors (joint venture)
Bellamy Brothers, Inc., Ellenwood, Georgia and Phillips & Jordan, Inc.
Knoxville, Tennessee

Steel Fabricator
Carolina Steel Corporation
Greensboro, North Carolina

Steel Erector
Bellamy Brothers, Inc.
Ellenwood, Georgia

Owner
U.S. Army Corps of Engineers
Savannah, Georgia

PRIZE BRIDGE 1984—
MEDIUM SPAN,
LOW CLEARANCE

NORFORK LAKE HIGHWAY BRIDGES 8

Baxter County, Arkansas

Twin spans solve the problem of increased traffic across the reservoir previously served by ferry boats. Their clean lines and lightweight superstructure of weathering steel are fashioned of two continuous-depth welded plate girders with floor beams and stringers. Steel proved to be lowest in cost and provided the best appearance. The lightweight girders are fixed to pier tops with pre-stressed anchors that permitted framed bent-type piers to be constructed above water, and without falsework.

Designer
Howard Needles Tammen & Bergendoff
Kansas City, Missouri

General Contractor
Massman Construction Company
Kansas City, Missouri

Steel Fabricator
Kansas City Structural Steel Corporation
Kansas City, Kansas

Steel Erector
Vogt and Conant Southwest Corporation
Little Rock, Arkansas

Owner
U.S. Army Corps of Engineers
Little Rock, Arkansas

PRIZE BRIDGE 1984—
RECONSTRUCTED

LIBERTY BRIDGE 9

Pittsburgh, Pennsylvania

Deemed the most extensive bridge reconstruction project undertaken in the state, the structure required both involved design tasks and intricate jacking schemes. The complex project included repairs to the superstructure, deck replacement and widening and replacing 18 of 20 main truss bearings. Imaginative engineering transformed an obsolete 2,663-ft long bridge with daily traffic of over 50,000 vehicles into an efficient transportation structure.

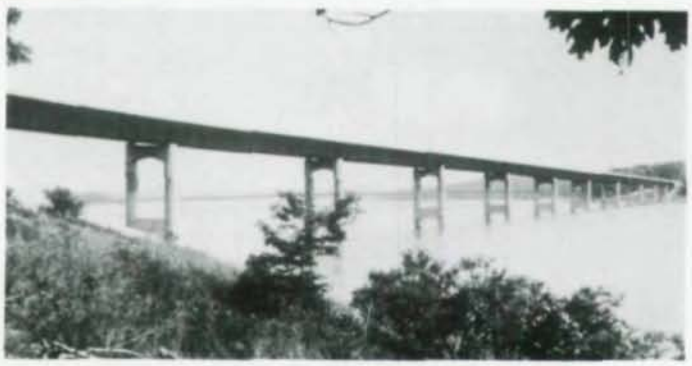
Designer
Salvucci & Associates, Inc.
Pittsburgh, Pennsylvania

General Contractor/Erector (joint venture)
Dick Corporation/Dick Enterprises
Pittsburgh, Pennsylvania

Steel Fabricator
Atlas Machine and Iron Works, Inc.
Gainesville, Georgia

Owner
Pennsylvania DOT
Pittsburgh, Pennsylvania

7



8

9

PRIZE BRIDGE 1984—
MEDIUM SPAN,
HIGH CLEARANCE

KY80 over ROCKCASTLE RIVER 10

Pulaski-Laurel Counties, Kentucky

The owner specified steel plate girders for this structure. Five framing alternates were considered, with a five-span continuous structure proving the best. Its design avoids cluttering the visual horizon and keeps approach fill above the high water level. Longitudinal and transverse stiffeners on the interior web permit minimal use of exterior stiffeners. The jurors point to a well-proportioned span with details that present a striking effect.

Designer
Kroboth Engineers, Inc.
Lexington, Kentucky

General Contractor
R. R. Dawson Bridge Company
Lexington, Kentucky

Steel Fabricator
High Steel Structures, Inc.
Lancaster, Pennsylvania

Owner
Kentucky DOT
Frankfort, Kentucky

SPECIAL AWARD 1984—
MOVABLE SPAN

VETERANS MEMORIAL BRIDGE 11

Kaukauna, Wisconsin

Extensive engineering investigation determined a lift span was the most economical one for this U.S. Canal crossing, based on site conditions and constraints. Welded steel fabrication attained the clean lines and simple details required on the overhead construction. Cost-effective welded steel box sections form all main towers and bracing members. A transverse Vierendeel strut simplifies design and fabrication of the towers that contain machinery and controls. The jurors acclaimed the clean lines and sculpture-like simplicity.

Designers
Ayres Associates, Eau Claire, Wisconsin, and Harrington & Cortelyou, Inc.
Kansas City, Missouri

General Contractor/Erector
Lunda Construction Co.
Black River Falls, Wisconsin

Steel Fabricator
Phoenix Steel, Inc.
Eau Claire, Wisconsin

Owner
City of Kaukauna, Wisconsin

SPECIAL AWARD 1984—
RAILROAD

THE GULF BRIDGE 12

Lockport, New York

A 12-span single track ballast steel structure, the 965-ft bridge crosses a deep valley known as "The Gulf." The structure, which required piers 86 ft above grade, was designed for Cooper E-72 loading of the forces of a 13,000 ton train. Twin girder spans, 80-ft long, are supported by
2- and 4-legged A-frames of 36-in. pipeline. These unique piers rest on caissons up to 97 ft deep. The result—a pleasing structure created by a high-tech solution of an older technology.

Designer
Bechtel Civil & Minerals, Inc.
San Francisco, California

Consultant
Ralph Whitehead & Associates
Atlanta, Georgia

General Contractor/Erector (superstructure)
Cives Steel Company, Conklin, New York

General Contractor (substructure)
Lane Construction Corp.
Meriden, Connecticut

Owner
Somerset Railroad Corporation
Binghamton, New York

10

11

12

SPECIAL AWARD 1984—
SPECIAL PURPOSE

R.C.C. PIPERACK BRIDGE

Catlettsburg, Kentucky 13

Two arch ribs support a piperack to carry petroleum over a multi-track right of way, a private spur and an access road—and solve an age-old problem. The box-girder ribs and secondary members were fabricated of A572 high-strength steel, with fireproofing added. The slender circular form, erected in three weeks without falsework, creates a visual link to a highly industrialized site to produce an unusual treatment of a common structure not normally attractive.

Designer
Columbus Engineering Consultants, Ltd.
Columbus, Ohio

General Contractor/Owner
Ashland Petroleum Company
Ashland, Kentucky

Steel Fabricator
Mid States Steel Products Co.
Lexington, Kentucky

SPECIAL AWARD 1984—
MEDIUM SPAN,
HIGH CLEARANCE

BONANZA-WHITE RIVER BRIDGE 14

Uintah County, Utah

It fits so well into the landscape that few realize it is a bridge, were it not for railings. Designed and erected in nine months, it met strict deadlines. This three-span welded plate bridge, designed to AASHTO HS-30 loads, carries heavy equipment to a shale mine. Haunched weathering steel girders fit aesthetically with the area's geology. The jurors considered the bridge spectacular for its height, span and loading.

Designer
Horrocks Engineers
American Fork, Utah

Consultant
D. Allen Firmage
Provo, Utah

General Contractor
W. W. Clyde Construction
Springville, Utah

Steel Fabricator/Erector
Utah Pacific Bridge & Steel Company
Lindon, Utah

Owner
Uintah County, Vernal, Utah

SPECIAL AWARD 1984—
RECONSTRUCTED

ELLICOTT CREEK BRIDGE

Buffalo, New York 15

Deck timbers, steel girders and stringers of this bridge, built around 1890, finally succumbed to time and the elements—and could no longer carry heavy maintenance trucks and utility vehicles. Using salvaged timbers and steel, the contractor rebuilt the structure for less than $20,000. Two 9-ft o.c. main girders were set in refurbished stone abutments. Now capable of carrying modern loads, the bridge stands as a perfect example of updating old structures to new needs.

Designer
Joseph Freeman, P.E.
Buffalo, New York

General Contractor/Steel Fabricator
J. W. Lintner Construction, Inc.
Williamsville, New York

Owner
Park Country Club of Buffalo
Williamsville, New York

13

14

15

AWARD OF MERIT 1984—
GRADE SEPARATION

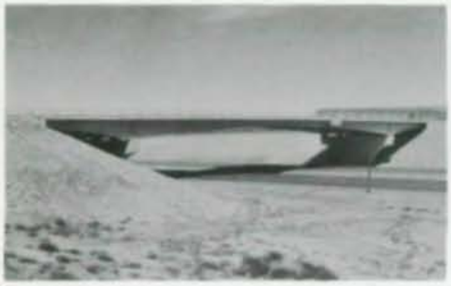
ALGODONES INTERCHANGE

Algodones, New Mexico

Designer/Owner
New Mexico State Highway Department
Santa Fe, New Mexico

General Contractor/Erector
A. S. Horner, Inc.
Littleton, Colorado

Steel Fabricator
The Midwest Steel & Iron Works Company
Denver, Colorado



AWARD OF MERIT 1984—
MEDIUM SPAN, HIGH CLEARANCE

LOCUST STREET VIADUCT

Milwaukee, Wisconsin

Designer
Bureau of Bridges & Public Buildings
Milwaukee, Wisconsin

General Contractor/Erector
Lunda Construction Company
Black River Falls, Wisconsin

Steel Fabricators
Hartwig Manufacturing Corporation
Wausau, Wisconsin, and
Phoenix Steel, Inc.
Eau Claire, Wisconsin

Owner
City of Milwaukee, Wisconsin

AWARD OF MERIT 1984—
LONG SPAN

McNAUGHTON BRIDGE

Pekin, Illinois

Designer
TEC Engineers, Ltd.
Northlake, Illinois

General Contractor/Steel Fabricator
Bristol Steel & Iron Works, Inc.
Bristol, Virginia

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

Owner
Div. of Highways, Illinois DOT
Springfield, Illinois

AWARD OF MERIT 1984—
SHORT SPAN

WHITECHUCK RIVER BRIDGE

Mount Baker/Snoqualmie National Forest, Washington

Designer
Federal Highway Administration
Denver, Colorado

General Contractor/Erector
W. C. McKasson, Inc.
Lilliwaup, Washington

Steel Fabricator
Fought & Company
Tigard, Oregon

Owner
U.S.D.A. Forest Service, Pacific NW Region
Portland, Oregon

AWARD OF MERIT 1984—
RAILROAD

MARTA AERIAL STRUCTURE over RAILROADS-CS310

Atlanta, Georgia

Designer
Anderson-Nichols & Company, Inc.
Boston, Massachusetts

General Contractor
Moseman Underground Continental
Heller, JV
Redding, California

Steel Fabricator
Carolina Steel Corporation
Greensboro, North Carolina

Owner
Metropolitan Atlanta Rapid Transit Authority
Atlanta, Georgia

AWARD OF MERIT 1984—
VIADUCT

UNION PACIFIC OVERPASS

Cheyenne, Wyoming

Designer
Bridge Design Branch, Wyoming State
Hwy. Dept., Cheyenne, Wyoming

General Contractors/Erectors
Engineered Structures of Wyoming,
Cheyenne, and
Sletten Construction Company
Great Falls, Montana

Steel Fabricators
Carolina Steel Corporation
Greensboro, North Carolina, and
The Midwest Steel and Iron Works, Inc.
Denver, Colorado

Owner
Wyoming State Highway Department

AWARD OF MERIT 1984—
MEDIUM SPAN, LOW CLEARANCE

GREEN RIVER BRIDGE

Uintah County, Utah

Designer
E. W. Allen & Associates
Salt Lake City, Utah

General Contractor
W. W. Clyde & Company
Springville, Utah

Steel Fabricator/Erector
McNally Mountain States Steel Co.
Lindon, Utah

Owner
Uintah County Commission, Vernal, Utah

AWARD OF MERIT 1984—
SPECIAL PURPOSE

NORTH BIKEWAY BRIDGE over GREATER MIAMI RIVER

Dayton, Ohio

Designer
Lockwood, Jones & Beals
Dayton, Ohio

General Contractor
Miller-Valentine Corporation
Dayton, Ohio

Steel Fabricator
Mound Steel Corporation
Springboro, Ohio

Owner
The Miami Conservancy District
Dayton, Ohio

AWARD OF MERIT 1984—
GRADE SEPARATION

MERRITT PARKWAY over ROUTE 8

Trumbull, Connecticut

Designer
Seelye Stevenson Value & Knecht, Inc.
Stratford, Connecticut

General Contractor
Arute Brothers, Inc.
New Britain, Connecticut

Steel Fabricator/Erector
The Standard Structural Steel Company
Newington, Connecticut

Owner
Connecticut DOT
Wethersfield, Connecticut

AWARD OF MERIT 1984—
MEDIUM SPAN, LOW CLEARANCE

LEATHERWOOD FORD

Big South Fork National River & Recreation Area, Tennessee

Designer
Kroboth Engineers, Inc.
Lexington, Kentucky

General Contractor/Erector
Elmo Greer & Sons, Inc.
London, Kentucky

Steel Fabricator
Gamble's Inc.
Montgomery, Alabama

Owner
U.S. Army Corps of Engineers
Nashville, Tennessee

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

Address Correction Requested