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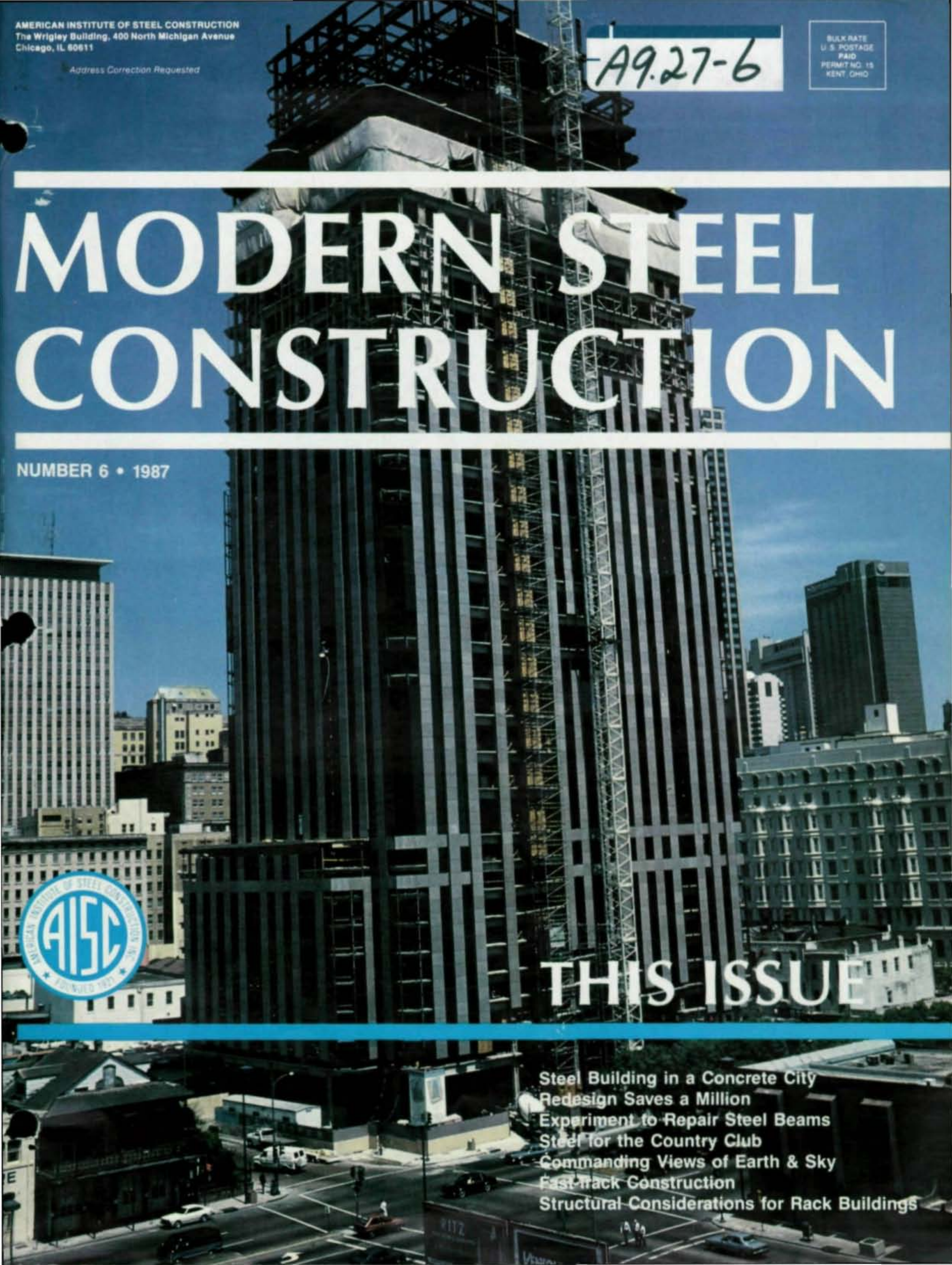
MODERN STEEL CONSTRUCTION

NUMBER 6 • 1987



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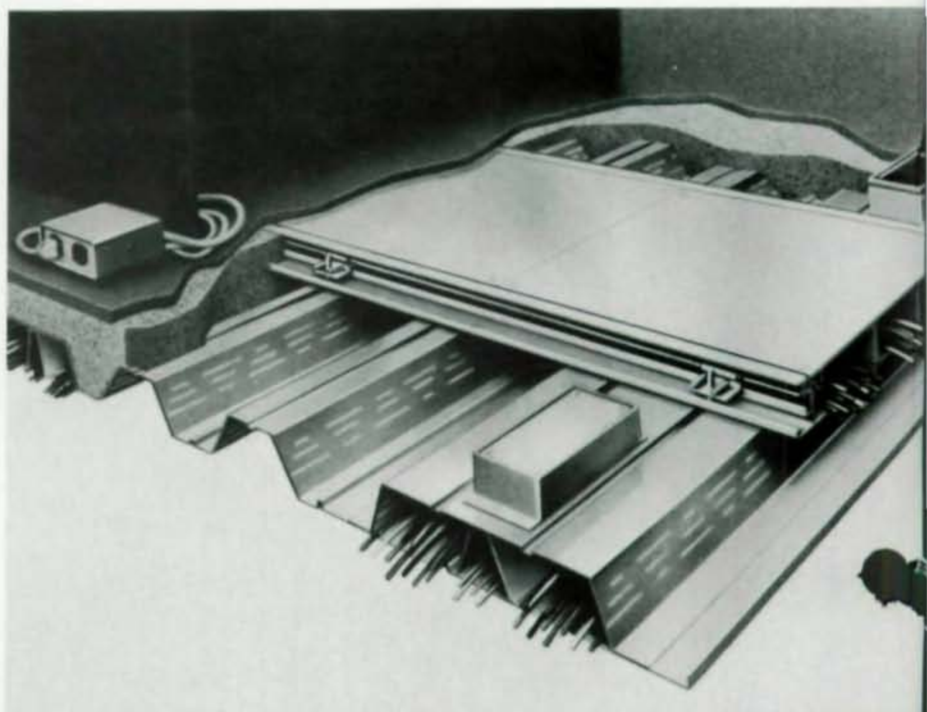

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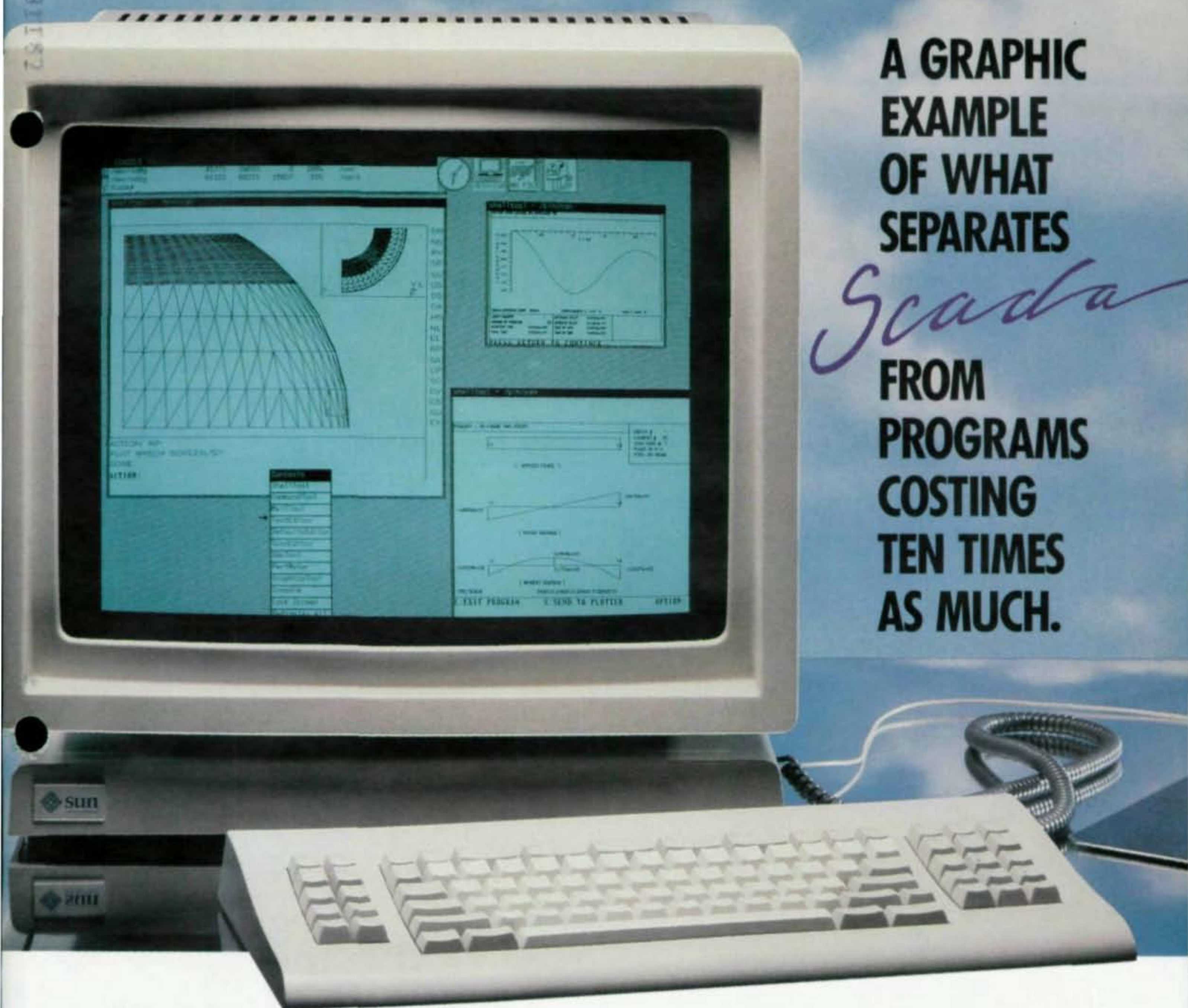

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MODERN STEEL CONSTRUCTION

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ON OUR COVER:
LL & E Tower,
New Orleans. See p. 5.

LL&E TOWER

A Steel Building in a Concrete City

by James M. Notch

Abstract
This paper documents the structural design and analysis as it evolved in the planning of the new LL&E Tower in New Orleans, La. The building, a project of Joseph C. Canizaro Interests, will serve Louisiana Land and Exploration Co., as its major tenant. The 785,000-sq. ft tower has 36 levels above grade and is capped with a two-story high sloped parapet. Hurricane wind force resistance for the bronze colored stone and glass tower is provided by a recently developed concept known as the "staggered-tree beam system." The tree beam modules, consisting of a wind girder fitted with intermediate stub column pieces to improve stiffness, conform to the building's architectural shape by shifting plan locations at various levels. The interacting system of tree beam modules work together to create an efficient, all steel, partial-framed tube system. Total steel framing unit weight was approximately 15.50 lb./sq. ft.

This feature documents the structural system conceived and used on the new LL&E Tower in New Orleans, La. The $80-million project of Joseph C. Canizaro Interests, will provide space for Louisiana Land and Exploration Co., its major tenant.

Local economic factors and tradition have dictated that major buildings in New Orleans are to be built of cast-in-place concrete construction. The owner challenged the structural engineers to counter the trend and develop a structural steel system for the project. The steel system would assist the owner in attaining a very short construction schedule. The engineers were challenged in that the structure must be developed with sufficient efficiencies so it could be built as competitively as local conventional concrete systems. This feature provides an overall review of the structural system on this project and specific details regarding its design.

General Description

On Poydras Street in downtown New Orleans, the 785,000-sq. ft tower, with 36 levels above grade, rises 482 ft above street level. The structure's first-floor lobby/retail level is separated from its 26 office levels by a nine-level, ramped auto parking structure.

The classicized tower design has overall plan dimensions of approximately 215 x 150 ft at its base, and tiers to a 115 x 115 ft floor dimension at the top. The tiered sides of the building not only provide the owner with unique floor plans matched to his needs but also panoramic views of the Crescent City.

The tiered building design, in conjunction with column constraints imposed by the parking garage, presented a challenge to the structural engineer to provide lateral force resistance for the tower. A further challenge was the time constraint on the construction schedule. The short construction schedule pointed to the need for a steel structural system. However, steel office tower construction in New Orleans has been virtually non-existent. The local construction scene centers around cast-in-place concrete structures. The structural engineer broke with the local tradition and developed a steel system which could be built on a competitive basis with a concrete cast-in-place system. The challenge was met successfully by using an innovative hybrid system employing several structural concepts. And total steel framing unit weight was limited to approximately 15.50 lbs./sq. ft.

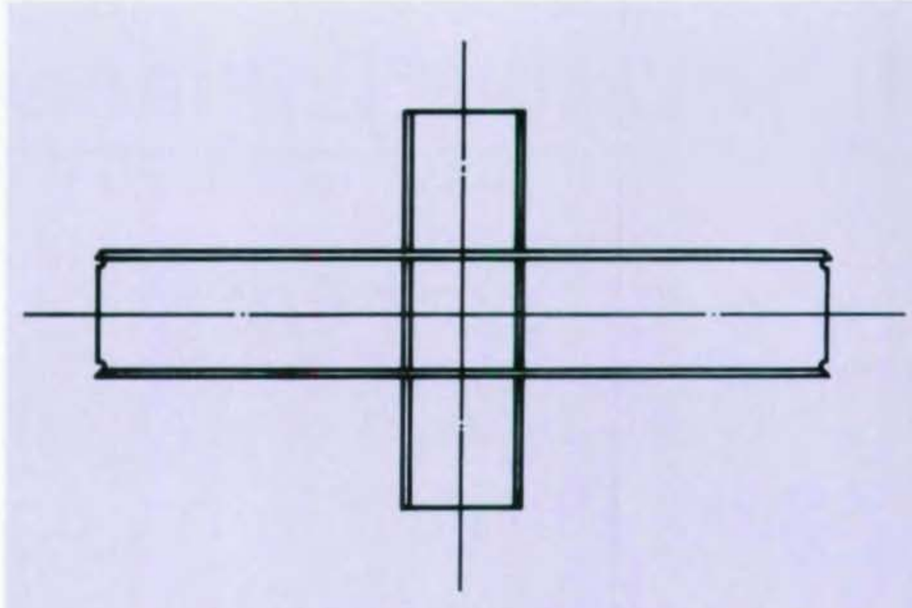


Fig. 1 Single-stem, tree-beam module

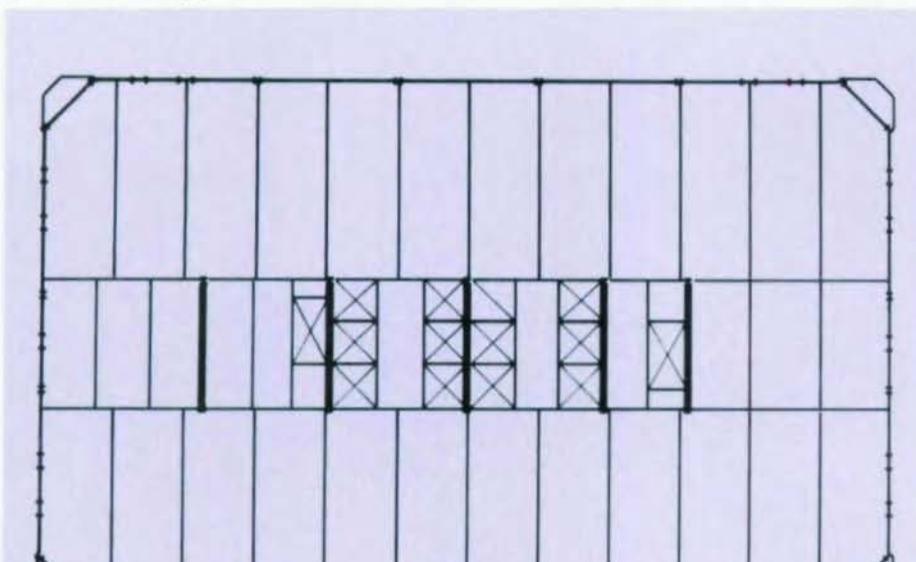
Lateral-resistant System

The architectural core layout was configured so it could incorporate several lines of inverted K-truss bracing. Five lines of diagonal bracing were provided for resistance of wind on the building's broad exposure. To simplify fabrication, the inverted K-truss type diagonal bracing in the core area is 2L or 4L struts, lapped at each end and bolted to gusset plates. The horizontal strut is a double channel with the gusset plates sandwiched and bolted in between at each end and the center. Gusset plates were fillet-welded to the core columns as required. At the lower levels, the magnitude of forces dictated that cover plates be added to the 4L assembly to create a box section. The cover plate was field-welded to facilitate bolt installation at ends of the bracing.

A single, two-bay line of diagonal bracing was used for wind resistance on the narrow exposure. It was located in such a manner that it intersected the extreme end column of three of the brace lines oriented in the perpendicular direction. In this manner, it activated additional stiffness and served as a large cantilevered channel with respect to wind on the narrow exposure. Since the central shear truss' least depth was limited to only 26.75 ft, the core bracing system acting alone was not adequate, nor desirable. Some form of very stiff frame action was required to control building sway. Because of the building's tiered layout and the desire for open lease areas, conventional moment framing was neither adequate nor attainable. The solution was to develop a unique system using prefabricated, tree beam modules.

The modular construction method with tree beams has been described in several papers previously published in *Modern Steel Construction* by this author. To enhance the stiffness and strength of conventional wind girders, the tree-beam concept was developed whereby short vertical stub columns at mid-span are added to heavy horizontal wind girders (Fig. 1). The stub columns force an intermediate inflection point in the member, thus greatly increasing its stiffness and strength (Figs. 2,

Typical floor framing plan



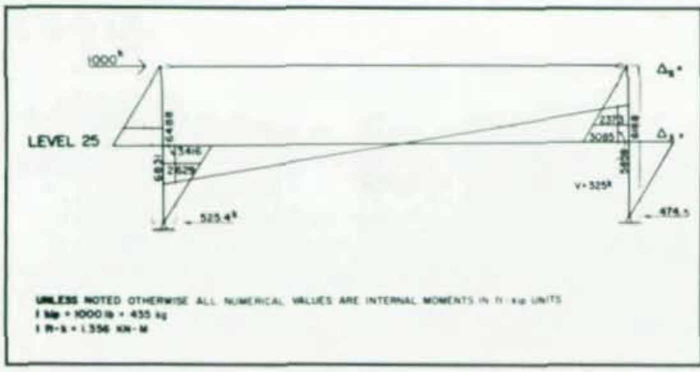


Fig.2. Frame action with conventional wind girder

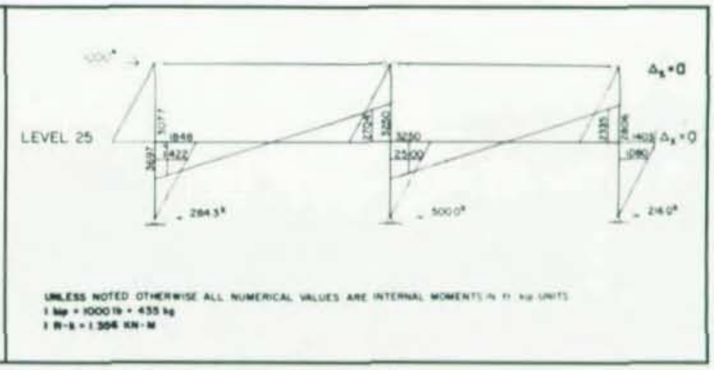


Fig. 3. Frame action with tree-beam module

3). Since the stub pieces perform only a flexural function, and shear truss/frame interaction is dominant in the intermediate height range of the tower, they may be easily deleted at the lower levels and top levels to create expansive, open architectural spaces.

Since the tree-beam elements behave primarily as flexural elements subject to shear racking, it is possible to shift the location of frames containing groups of the modules as the building tiers inward. At levels 2 through 10 (Fig. 4), the tree-beam frames were located at the extreme ends of the tower. The vertical stub column pieces were positioned directly behind 5-ft wide vertical stone cladding bands. As the plan changes from auto parking to office occupancy (Fig. 5), the tree-beam frames shift inward 15 ft. To remain back of the stone facade, which also moves inward in this position, new columns are activated with regard to axial shortening due to wind. By using additional columns for axial stiffness, the framing system's overall stiffness is augmented.

At the typical office levels, tree-beam modules were added to the end bays on the longitudinal faces of the tower. These modules tie into the terminal end columns of the side frames to simulate a large channel or C-shape assembly in plan. These channels cantilever upward from the tower's 10-story base. This configuration created a very stiff assembly of frames which, through truss/frame interaction with the core bracing, minimized building drift under the most severe hurricane wind loading conditions.

At level 28, the tower continues a series of setbacks. These setbacks result in the

Fig. 4. At levels 2-10 , tree-beam frames are at ends of tower.

Fig. 5. As building changes from parking to office, frames shift inward 15 ft.

Single line of diagonal bracing used for wind resistance.

end tree-beam frames again shifting location. In the process of re-configuring the plan at levels 28, 31, 35 and at the two-story high cornice/parapet, many columns were terminated and new columns started. Under conventional framing techniques, the new columns would have been located on deep, steel-plate transfer girders. However, since the tree-beam frames were present for wind resistance, they were also facilitated for use in transferring the gravity loads. Iteratively analyzed using stage construction simulation techniques, the assembly of tree beams act as a massive Vierendeel truss. In this way, steel premiums necessary to accomplish the floor transfers were minimized.

Structural Framing Details

Columns were typically W14 rolled sections. The heaviest loaded columns ranged in size from cover-plated W14 x 730 at the base to W14 x 90 at the top. In the lowest levels, wherever design loads exceeded the column capacity, cover plates were connected from flange tip to flange tip. All columns beared on milled steel base plates which were shipped loose. Columns were anchored by using anchor bolt boots attached to the sides of the column shafts. The largest base plate weighed 10,591 lbs. (56 in. x 58 in. x 11½ in.). A 3-in. thick layer of non-shrink grout, 7,500 psi strength at 28 days, was provided between the bottom of the steel base plate and the top of the pile cap foundation. The base plates were flow-grouted from the side of the base plate assemblies. No grout holes were needed.

Column-to-column splice connections adjacent to tree-beam modules were made midway between spandrel beams at points of theoretical minimum moment. Typical flange welds were partial penetration to a depth of $\sqrt{T/6} + 1/8$ in. where T = column flange plate thickness. Weld depth was increased as required for columns with calculated uplift forces. Welds were also increased at isolated locations to accommodate shifts in the inflection point of the moment diagram. All bearing surfaces were milled. Column web-to-web connections were partial-penetration welded. Column web stiffeners aligning with the spandrel beam flanges were provided to meet requirements of strength or, as in most conditions, to stiffen the beam column joint and restrain panel zone flexibility. In selected areas, where interstory drift was minimal, horizontal stiffeners (continuity plates) were not used.

At core wind frame column bases, uplift forces due to lateral loading were significant. Uplift forces were resisted by anchor bolt assemblies utilizing 2-in. dia. A354 Gr. BD bolts, which were embedded deep into the pile caps and restrained from pull-out by large, stiffened washer-plate assemblies. Most tree-beams used W36 or W30 sections.

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

The tree-beam elements were fully moment-connected using field welding to the columns at each end. The stub-column, piece-to-stub column piece connection joints were gapped and non-bearing with field bolt-up using 1-in. A490 bolts with double splice plates. For horizontal girder members, rolled sections were used which varied in size from W36 x 300 to W36 x 135. The vertical stub-column piece elements were made from rolled W36 shapes. Stub column piece sizing was slightly less than its respective horizontal beam piece. The vertical stub-column elements were shop moment-connected to the horizontal member. Because of high panel zone stresses, thick doubler plates were added

between flanges in some panel zone areas.

Gravity Framing System Concepts

The typical floor construction is 6¼-in. thick composite, metal deck slab construction (3¼-in. lightweight slab on 3-in. metal deck). The typical deck was 16 ga., 36 in. wide with a 12-in. rib spacing. Phosphatized painted deck was used over office levels and galvanized deck in garage areas. The deck, typically spanning 15 ft, was supported by W21 rolled sections.

Spanning the approximately 40 ft from perimeter frame to central core area, typical beams were designed to act compositely with the concrete slab by means of shear studs, field installed through the metal deck. A shop camber was specified on all long purlins to compensate for the deflection of the beam under the weight of wet concrete, thus providing a constant thickness and level floor system after pouring of the slab. Because of metal deck deflection between beams, the composite slab was thicker in the area between beams. This additional concrete ponding weight was considered in the gravity framing and deck design. Nominal 6 x 6-W1.4 x W1.4WWM was typically provided in the floor slab. For crack control, negative moment top bars and 6 x 6-W2.9 x W2.9WWM was used at garage levels. This slab reinforcement was augmented in several areas of high diaphragm stress, as required. U-shaped rebar ties were provided at the perimeter of the building slab to provide a mechanical tie between the shear connectors located on the spandrel beams and the floor diaphragm. This mechanical tie provided for bracing of the columns into the floor diaphragm as well as transfer of wind shears into the diaphragm. The U-bars served the additional purpose of supporting the stone facade located at the end of the slab cantilever. □

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New York, New York

Technical Architect
Perez Associates
New Orleans, Louisiana

Structural Engineers
Ellisor & Tanner, Inc. (superstructure)
Houston, Texas
Morphy, Makofsky and Masson, Inc. (foundation)
New Orleans, Louisiana

General Contractor
Gervais F. Favrot Co.
New Orleans, Louisiana

Owner/Developer
Joseph C. Canizaro Interests (joint venture)
New Orleans, Louisiana

James M. Notch, M., ASCE, is president of The Datum/Moore Partnership, Irving, Texas. Notch, a professional member of AISC, was vice president, Ellisor & Tanner, Houston, during design of this project.

Redesign in Steel on Emlenton Bridge Saves a Million

by Louis P. Schwendeman

Preface

Carl Angeloff, regional bridge engineer, AISC Marketing, Inc., first targeted this project for a redesign as a steel contractor's alternate, after a routine visit with design engineer in August 1986. Subsequent investigations convinced the AISC Marketing staff this project was an excellent candidate for a contractor's alternate steel design.

After receiving clearances from PDOT that a contractor's alternate would be accepted on the project, AISC Marketing developed a Preliminary Design Study. This study was made available to all interested fabricators and contractors.

The following outlines the culmination of a 5-month effort to successfully redesign and bid a steel contractor alternate on the Emlenton Bridge.

Clean, uncluttered steel frame of Emlenton Bridge superstructure

In recent years, a lively competition has developed between the prestressed concrete and steel industries for the design and construction of highway bridges. For a time several years ago, most short spans under 100 ft and many intermediate and longer span bridges were built of concrete. Recently, however, steel bridges are making a dramatic comeback when the construction contracts are bid for the longer span bridges on the basis of alternate designs in concrete or steel.

There are a number of reasons for the current improved position of steel bridges:

1. Steel fabrication costs within the industry have been reduced.
2. Steel details have been simplified.
3. Load factor design of deck slabs reduces slab thicknesses and resultant costs and dead loads.
4. Use of the finite element method of analysis distributes and reduces live loads to the stringers via the deck slab.
5. Improved stay-in-place metal forms permit wider spacing of girders.
6. Load factor design results in lighter girder sections with wider stiffener spacings.
7. AASHTO specifications permit the reduction or elimination of lateral bracing in short and intermediate length spans.
8. American Welding Society (AWS) specifications permit reduction in size of fillet welds where governed by maximum thicknesses of joined plates.

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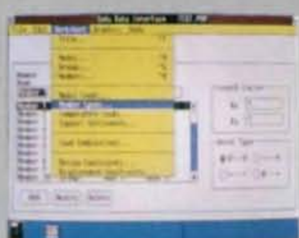

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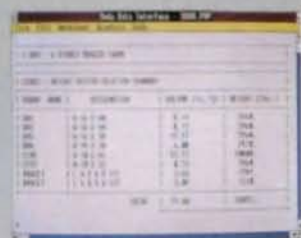

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come in the market. The structure was originally designed for the Pennsylvania Department of Transportation as a six-span prestressed concrete I-beam bridge 627 ft long with spans of 103 ft-7 in., four at 104 ft-11½ in. and 103 ft-7 in. The four lines of beams were designed as six composite simple spans under dead load and as six-span continuous units under live plus impact loads after concrete continuity diaphragms were poured in the deck and at the beam joints over the five piers. The two-lane bridge measured 36 ft curb-to-curb and 38 ft-6 in. out-to-out of 1 ft-3 in. wide concrete safety curbs.

In November 1986, the department released for bidding the plans and documents for the concrete beam bridge with provisions for alternative concrete or steel designs, with a bid opening set for Dec. 18, 1986. In early December, Trumbull Corporation, Pittsburgh, Pa., requested HDR-Richardson Gordon, Inc., Consulting Engineers, Pittsburgh, Pa., to investigate if a steel girder bridge alternate might be competitive. Since the specification required the center span of any alternate steel girder bridge must be 240 ft long and the 38 ft-6 in. width and 627 ft overall length must be retained, it was determined readily any new steel structure should have three continuous spans of 193 ft-6 in., 240 ft and 193 ft-6 in.

Using the tables in the USS *Composite Steel Plate Girder Bridge Superstructures Handbook* (Load Factor Design) for span lengths of 190 ft, 240 ft, 190 ft and using ASTM A588 steel, it was determined the superstructure steel would weigh approximately 35.5 lbs./sq. ft of deck. Since the new bridge was to be designed for HS25 loading instead of the HS20 loading used in the USS Handbook, and since splice material, bearings, expansion dams, weld material and bolts were not included in the steel weight in the Handbook, the steel weight was increased by approximately 10% to arrive at a steel weight of 39 lbs./sq. ft of deck, or a total steel weight of 470 tons.

This preliminary information was developed by the consultant within a few hours. Thus, they could have ample time to balance the cost increase in the steel superstructure with longer spans against the elimination of the cost of three piers in the shorter spans in the concrete design before making a decision whether to proceed with the steel design. Since the concrete piers supported on 42-in. dia. drilled-in concrete caissons were expected to be very difficult and expensive to construct, it did not take the contractor long to decide to bid the steel alternate.

The consultant next recommended that

Comparatively light steel girder sections swing into place.

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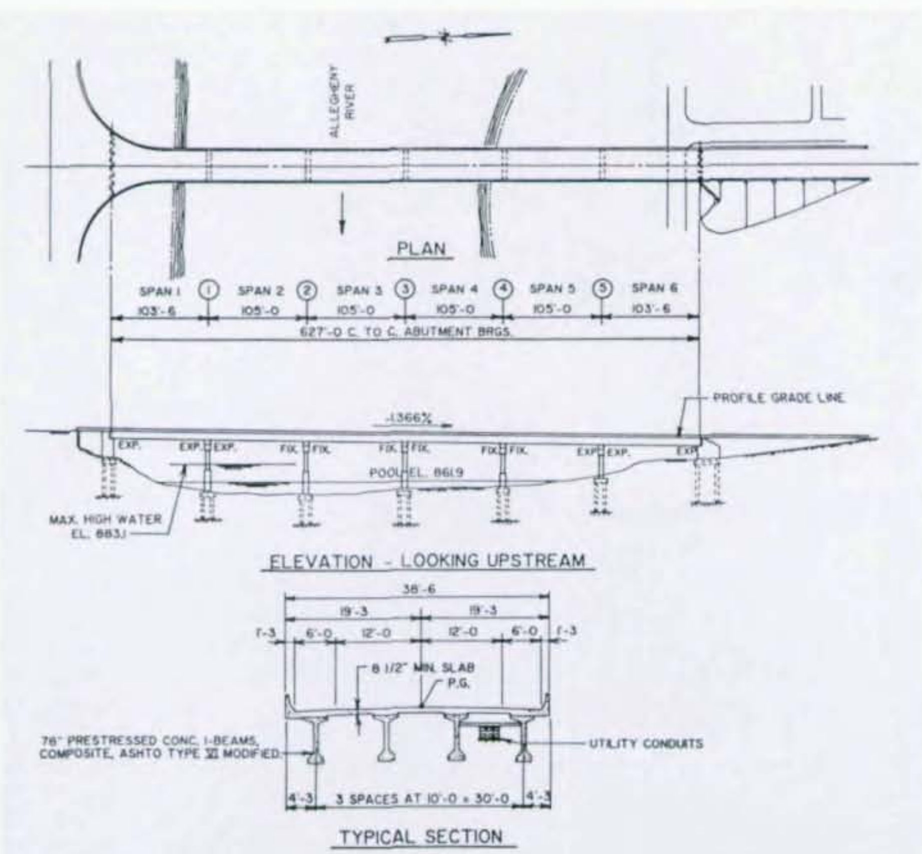
Trumbull request assistance from the Pittsburgh office of AISC Marketing, Inc., to substantiate the alternate steel design weights. AISC responded very promptly on Dec. 8, 1986, with two suggested solutions for the steel alternate design. One scheme was for a five-span continuous steel girder bridge; the second was for a four-span continuous structure. Both schemes showed four lines of girders at 10-ft centers—the same stringer spacings used in the original concrete design. These solutions indicated even more weight reduction in the steel than previously anticipated, and further substantiated the viability of an alternate steel bid. However, since the department had specified a 240-ft long main center span would be required for any redesign in steel, a three-span continuous bridge appeared most logical. Thus, the steel weight savings indicated by the AISC redesign studies with shorter spans could not be fully realized.

HDR-Richardson Gordon agreed with Trumbull Corporation to proceed with the further refinement of the steel structure design so final bidding figures could be developed. The AISC Marketing Computer Program SIMON proved invaluable for this review. A steel weight of approximately 500 tons was finally agreed upon for bidding. This compared very favorably with the 513 tons of steel required for the subsequent refined final design. This increase over the originally estimated 470 tons of steel may be attributed to the fact that because of the desire to retain the original stringer spacings and deck slab overhangs, the steel weight savings realized from the finite element analysis did not prove to be as great as anticipated.

At the bid opening on Dec. 18, 1986, Trumbull was low bidder at $3,500,000 for the steel alternate design. The next bid, also for a steel alternate, was $231,000 higher, while the third bid—which was for the original concrete design—was $1,038,000 above the low bid. The remaining three bids were also for the concrete designed bridge with prices ranging upward to the sixth bid, over $3,000,000 above the low bid.

A notice to proceed was given by the department in early January 1987. A conceptual steel redesign for the bridge was required within 30 days, with the new bridge specified to be opened to vehicular traffic before the end of 1987. Although the concrete deck slab in the alternate design remained at the original 8½-in. thickness, there was a substantial 15% reduction in the quantity of deck concrete from 856 CY to 725 CY. This reduction could be attributed mainly to the elimination of heavy concrete diaphragms at the piers which developed continuity in the concrete design after the simple span beams were erected.



Two-lane bridge measures 36 ft curb-to-curb

The concrete abutments and piers as originally designed required minimal redesign work for the alternate steel bridge because of the substantial reductions in dead loads from the deck slab and girders in the steel design. The total weight of the concrete beams alone was 1,655 tons, over three times heavier than the 513 tons of steel in the alternate bridge.

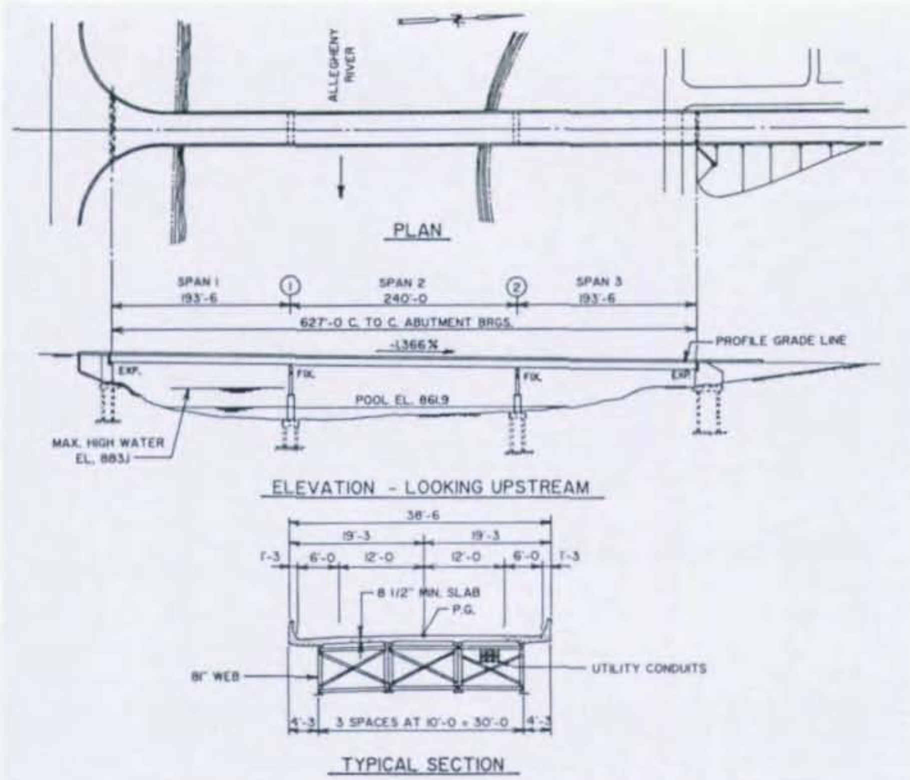
The total reactions from the superstructure to the abutments and piers in the long-span steel alternate design were so similar to those in the original short-span concrete design that very nominal changes were required. In fact, the structural concrete sections remained almost identical and the same number of caissons were retained for the piers and abutment 2 in the steel design. So the elimination of three piers in the steel alternate design was almost a clear-cut savings. Two fewer cofferdams, a 70% reduction in pier and abutment excavation, a 90% reduction in rip-rap protection at pier bases and 580 fewer lineal feet of 42-in. dia. caissons were also reflected in these savings. The very difficult drilling conditions through heavy dense sands, gravel and boulders in the stream eventually made the elimination of three piers all the more advantageous for the steel alternate design.

A finite element analysis was conducted to realize maximum efficiency in the steel stringer designs. To assist in this work, the services of BSDI in Coopersburg, Pa., were retained. The full advantages of the finite element analysis could not be realized because of several restrictions imposed by the original design specifications, such as:

1. Retain the original bridge length
2. Retain the original bridge width
3. Locate the river piers at 240-ft centers
4. The inventory rating using finite element analysis and HS25 loading shall equal or exceed an alternate inventory rating using conventional AASHTO load distribution factors for HS20 or the alternate military loading.
5. Make exterior stringers at least as heavy as the interior ones

Since it was also desirable that the original 10-ft stringer spacing and 4 ft-3 in. outside overhang be retained to simplify the design and details at the abutments and piers, less than maximum design efficiency resulted in which the exterior stringers are heavier than the interior stringers.

The final girder sections used for the four lines of stringers have 80-in. deep by ½-in. and 9/16-in. thick webs and maximum size flange plates 28 in. wide by 3-in. thick



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at the piers. The maximum total depth of the steel girder section is 87 in., which is 9 in. deeper than the 78-in. deep original prestressed concrete I-beam section, but within the vertical clearance limits. The structural steel for the girder flanges and webs is either ASTM designation A588 Gr. 50 or A572 Gr. 50. The structural steel for the two flared W36 x 150 stringers at the south end of the bridge, as well as all other sections, is ASTM A36, except for a few miscellaneous drainage and expansion dam details.

The paint specified for the structural steel is a three-coat shop-applied system consisting of an inorganic zinc primer, an epoxy intermediate coat and a urethane finish coat. Final field touchup is specified for damaged surfaces. Mechanically galvanized high-strength bolts requiring minimal field cleaning and touch-up painting were specified for all field connections. It is anticipated the paint system will protect the steel for at least 25 years with minimal touch-up repairs. The specified blue finish coat, contrasting with the grey concrete parapet and substructure, presents an attractive appearance.

The aesthetics of the structure are greatly enhanced by the elimination of three of the five piers required for the original prestressed concrete design. The two piers, rather than five, will also present less obstruction to stream flow and river traffic.

The redesign of the Emlenton Bridge as a continuous steel girder structure was completed in about six weeks after notice to proceed. The revised finished plans were submitted to the Pennsylvania Department of Transportation for final approval by Feb. 20, 1987. Few problems were encountered during approvals to delay the scheduled construction startup. Construction is currently on schedule, and it is anticipated will remain on schedule until the planned opening of the bridge in December 1987.

Only the driving of sheet piling for the cofferdam through the dense river bed and boulders at Pier 1 created a major construction problem which necessitated revising and reversing the erection scheme for the steel girders. Even though the final steel erection proceeded on schedule, the difficulties and excessive costs at the two river piers only further accentuated the advantage of eliminating the other three piers present in the original concrete design. In fact, the general contractor doubted his ability to meet the bridge opening date if he had to construct three more piers. He also envisioned extreme difficulties in transporting and erecting the deep 105-ft long prestressed concrete I-beams, each weighing 68 tons, if he had bid the original concrete design. Photos show the comparatively light steel girder sections duing erection and the clean uncluttered steel framing after the completion of the superstructure construction.

Although the general contractor had to assume the added cost of the steel redesign, including $5,000 to cover additional department engineering costs, he remains comfortable with his low bid for the steel alternate design—over a million dollars below the low bid for the original concrete design. This proves that steel bridges remain highly competitive, and actually much more versatile than concrete bridges, in meeting the variable construction conditions present at each site. When steel is used, there are no limitations on span lengths. And with its more efficient load-carrying capacity and comparable lighter weight than concrete, steel members develop fewer transportation and erection problems. Aesthetically, the finished steel bridge presents slimmer, cleaner lines than its bulkier, heavier competitor.

The main structural steel for this redesigned bridge for the Pennsylvania Department of Transportation, Dist. 1-0, was detailed by the WBE firm of Mellor Consultants, Inc., in Bethel Park, Pa. The steel girders and framing were fabricated by Reynolds Manufacturing Company of Avonmore, Pa. The fabricated steel was erected by Century Steel Erectors, Inc., of West Mifflin, Pa. As previously stated, the general contractor was Trumbull Corporation of Pittsburgh, Pa., and the redesign was made by HDR-Richardson Gordon, Inc., of Pittsburgh, Pa., with finite element assistance from BSDI of Coopersburg, Pa.

Designer/Structural Engineer
HDR-Richardson Gordon, Inc.
Pittsburgh, Pennsylvania

General Contractor
Trumbull Corporation
Pittsburgh, Pennsylvania

Steel Fabricator
Reynolds Manufacturing Co.
Avonmore, Pennsylvania

Owner
Pennsylvania Department of Transportation

Louis P. Schwendeman, P.E., is a retired principal of, and consultant to, HDR-Richardson, Inc., Pittsburgh, Pennsylvania.

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LSU Experiment Repairs Damaged Steel Beams

Steel ram swings against structure to simulate damage to overpass girders. Damage is assessed (r.).

Tempering heat applied with torch.

Louisiana State University engineers have successfully straightened in place a badly buckled, 1,500-lb. steel bridge girder with a heat-repair process that could save millions in highway maintenance dollars for Louisiana and the nation. The process, which makes damaged steel "as good as new," cuts down by as much as 90% the cost of total replacement of damaged overpass and bridge steel spans, according to R. Richard Avent, LSU professor of civil engineering.

Damaged steel girders can be straightened and repaired in a matter of days by applying tempering heat with an acetylene torch, thus minimizing road closure time, Avent said. In the LSU experiment, engineers straightened a two-foot deep, full-length girder damaged by a one-and-a-half ton steel ram swinging from a crane's 40-ft boom and packing a force equal to 10 tons. Avent said the ramming was designed to simulate the frequent accidental damage of highway overpass bridge girders by over-height trucks or steel damage to steel river bridges by barges or other vessels.

He said LSU research on the heat-tempering process so far shows that it not only straightens and repairs steel but also restores it to its original strength. Beams can be heat-repaired for a tenth of the estimated $100,000 cost of total replacement.

The technique was used to straighten and restore the strength of repeatedly damaged beams on an I-110 overpass bridge on the interstate near the governor's mansion in Baton Rouge. Avent continues, "The process has been more or less an art practiced by a few acetylene-welder craftsmen for some 50 years. Many states will not allow its use because of the current lack of documentation that structures repaired by the process are safe and reliable."

Supplying this scientific documentation is the objective of the LSU research, which is funded by the state Dept. of Transportation and Development and the LSU-head-

Repaired girder checked for alignment.

quartered Louisiana Transportation Research Center.

"That the process can restore the strength of damaged steel is hard for some people to believe," Avent said. LSU engineers have damaged and then repaired other smaller steel beams during the two-year research project. The heat-repaired steel was cut into sections, stress-and-load tested and then compared with known properties of similar undamaged steel. Tests showed no change in the properties of the steel, and the steel had remained as strong and as good as it was originally. The same testing procedures will be followed on laboratory analysis of the full-length beam.

The process requires heat-levels of no more than 1,200°F applied at certain locations and in specific patterns. The steel is allowed to cool briefly for thermal contraction in the opposite direction of the damage curve. As much as a one quarter-inch correction occurs between the initial applications, and the rate of correction slows as the girder straightens. □

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STEEL FOR THE COUNTRY CLUB

by Frederick M. Law

Steel framing was material of choice for Hawthorne Country Club addition.

Expansion of the Hawthorne Country Club, North Dartmouth, Mass., involved doubling the size of the main dining room on the first floor club, adding a second floor level deck above the main dining room and expanding the proshop and locker rooms at ground level.

The existing club house is a wood-framed building with a large number of interior columns and small floor-to-floor heights. To provide continuity, the new addition floor and ceiling elevations needed to match those of the existing club house. However, unlike the existing building, the new dining room should have as few interior columns as possible. Further, to take advantage of the splendid views of the golf course and the surrounding wooded river valley, the new main dining room would have exterior window walls of which were virtually all glass.

To meet these requirements, girder spans of 40 ft or larger were required. The total depth of construction (ceiling to floor above) was limited to a maximum of 27 in. at girder lines. In addition, no exterior (or interior) bracing was feasible, since it would interfere with the window walls.

Preliminary design computations confirmed wood framing would not fit into the 27-in. depth of construction without interior columns. Preliminary design computations, however, confirmed that conventional steel floor framing—steel beams, open-web joists and a reinforced floor slab on a metal form deck—fit nicely into the limited depth. Reinforced concrete framing, obviously, was not practical for this two-story building. Steel framing had an additional advantage in that it could be made into a three-dimensional frame to carry the lateral (wind) loads with relative ease. Therefore, steel was the clear choice for the expansion of the Hawthorne Country Club.

The typical connection detail used to

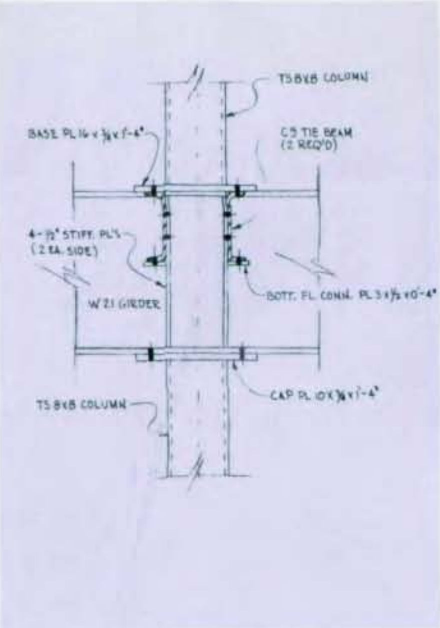


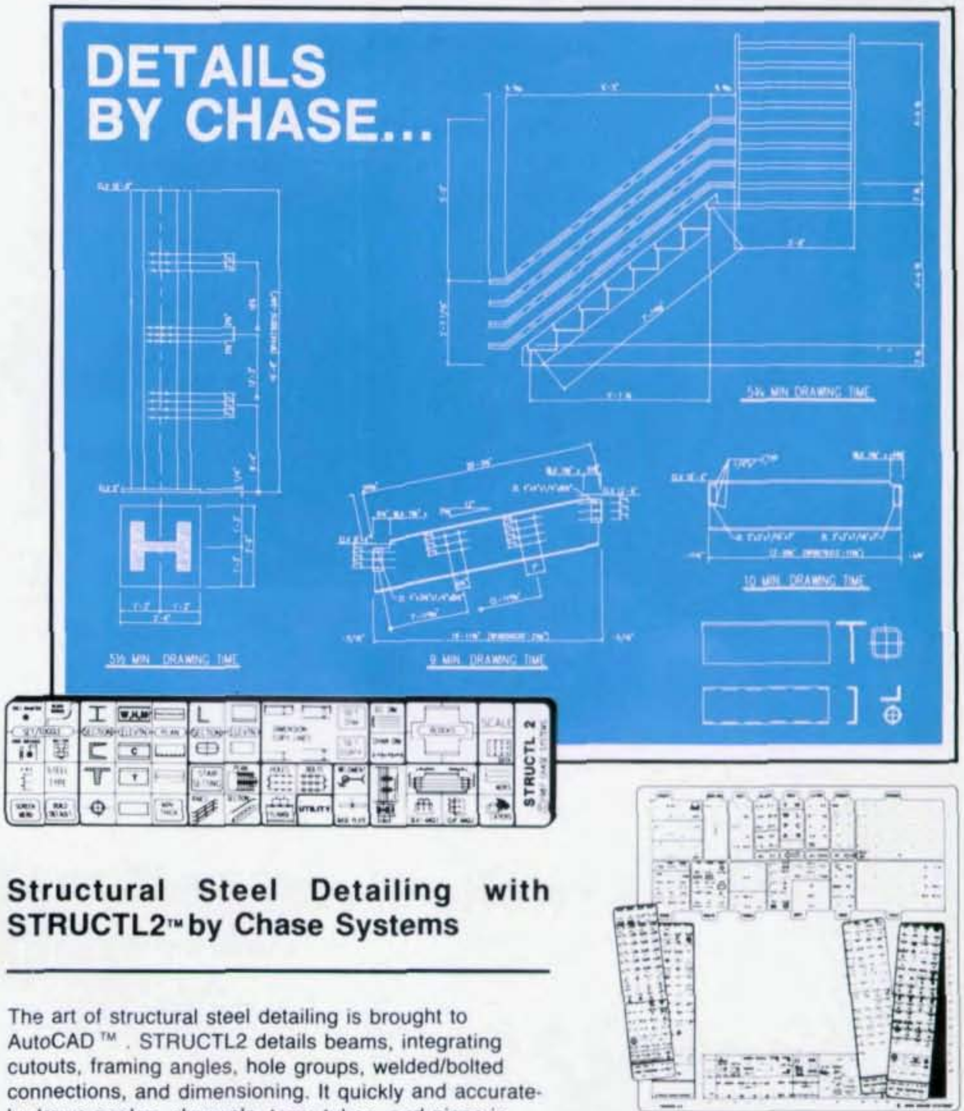
Fig. 1. Typical connection detail

create the steel frame is shown in Fig. 1. Note: the lateral (wind) load moments on the plane of the girders are transferred from columns to girders through the bolts of the cap and base plates of the columns. The lateral (wind) load moments perpendicular to the plane of the girders are transferred from the columns to double-channel tie beams through the bolts in the bottom flange connection plates and the base plates at the top flange. Even though the connections were designed to transfer lateral moments, the girders were designed as simple beams (unrestrained, free-ended construction) for gravity loads only. This design approach as been referred to as Type 2 Construction with wind-moment connections.

This apparently arbitrary design approach is actually very reasonable, if sufficient flexibility is provided in the connections to permit a shakedown of gravity moments. This is really a form of Type 3 Construction, semi-rigid framing (partially restrained construction). The design approach used for the country club addition is particularly advantageous for low-rise buildings. Wind moments are small and connections can be developed readily to carry these small moments. But connections required for full, rigid-frame action would be needlessly massive.

The total weight of steel required for Type 2 Construction with wind-moment connections is about the same as for Type 1 rigid-frame construction. However, fabrication costs are generally lower as a result of the less rigid connections required. Therefore, the total cost of the steel frame is generally less for Type 2 Construction

(continued)



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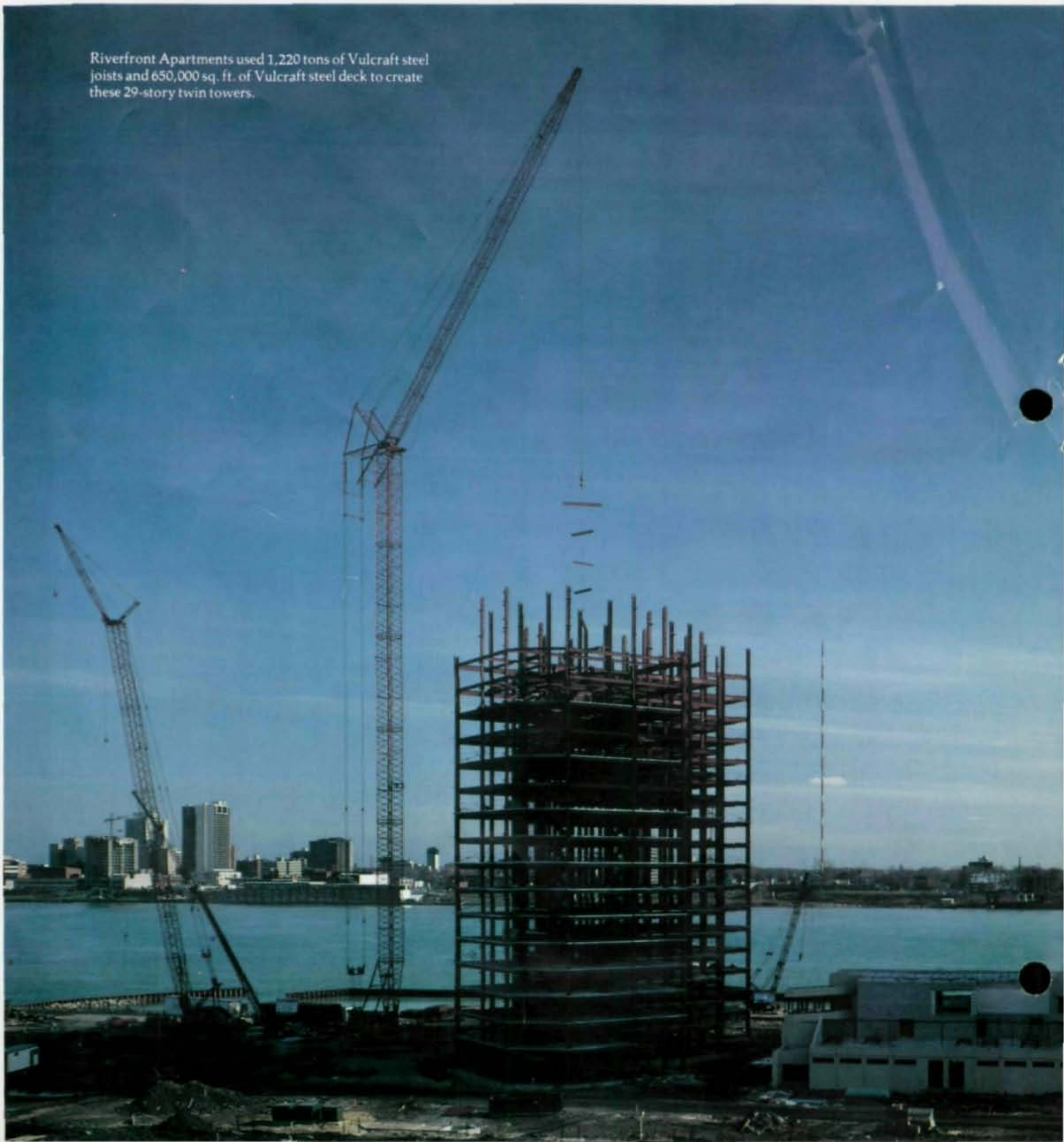

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Owner: Riverfront Associates/Builder: Barton Malow/Architect: The Gruzen Partnership/Structural Engineer: The Office of Irwin G. Cantor, P.C./Steel Fabricator: RCVNS Joint Venture (Ross Structural Steel Inc., Corvo Iron Works Inc., Vulcan Iron Works Inc., Noreast Erectors Inc., and Structural Steel Inc.)

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Scenic addition to country club

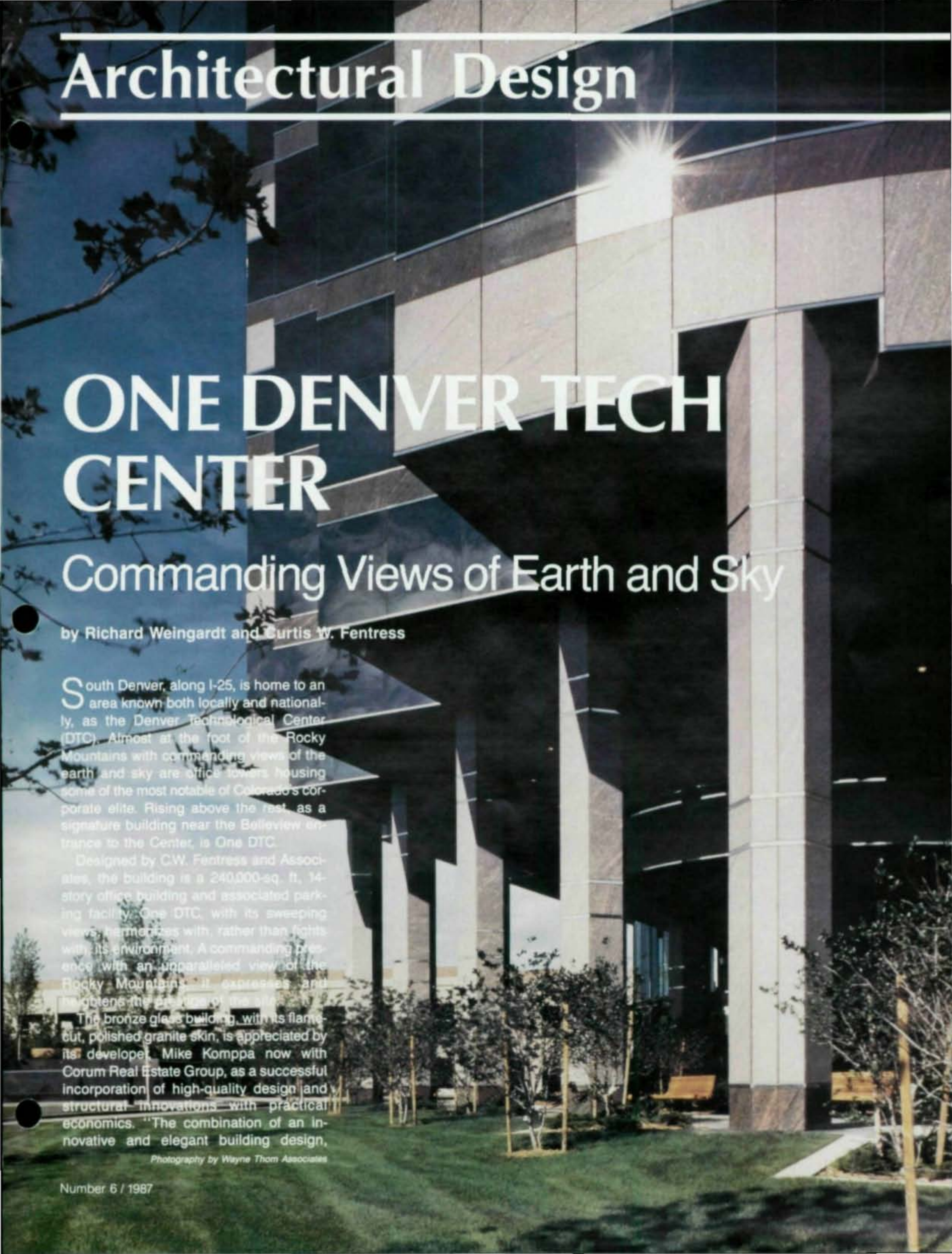
with wind-moment connections for low-rise buildings such as this country club addition.

Using a steel frame for the country club addition not only eliminated the need for bracing and interior columns, but also fit into the required limited permissible depth of construction. Finally, the addition was completed on time and within budget—all thanks to steel. □

Designer/Structural Engineer
Dr. Frederick M. Law
South Dartmouth, Massachusetts

Owner
Hawthorne Country Club
North Dartmouth, Massachusetts

Dr. Frederick M. Law, P.E., is principal in the structural engineering firm bearing his name in South Dartmouth, Massachusetts.



ONE DENVER TECH CENTER

Commanding Views of Earth and Sky

by Richard Weingardt and Curtis W. Fentress

South Denver, along I-25, is home to an area known both locally and nationally, as the Denver Technological Center (DTC). Almost at the foot of the Rocky Mountains with commanding views of the earth and sky are office towers housing some of the most notable of Colorado's corporate elite. Rising above the rest, as a signature building near the Belleview entrance to the Center, is One DTC.

Designed by C.W. Fentress and Associates, the building is a 240,000-sq. ft, 14-story office building and associated parking facility. One DTC, with its sweeping views, harmonizes with, rather than fights with, its environment. A commanding presence with an unparalleled view of the Rocky Mountains, it expresses and heightens the prestige of the site.

The bronze glass building, with its flame-cut, polished granite skin, is appreciated by its developer, Mike Komppa now with Corum Real Estate Group, as a successful incorporation of high-quality design and structural innovations with practical economics. "The combination of an innovative and elegant building design,

Photography by Wayne Thom Associates

open space and landscaped grounds, and highly desirable office amenities distinguishes it from others as the signature building of the Tech Center," Komppa comments.

High Water Table Challenge

The characteristics of this site posed a particular challenge to the design process. Architecturally, the challenges were threefold. Although the perfect location for the premier office location that One DTC was to be, the triangular shape of the site severely limited the amount of space with which to work. The project was made more challenging by the presence of a water table just a few feet below the surface. This fact alone shot the idea of an underground parking garage. Yet another challenge to overcome was the high noise level from I-25. The building and parking facility were sited in such a way as to leave room for tenants to stroll, picnic or just lounge in the noonday sun, and at the same time overcome all the obstacles.

Project designer Edward Balkin and Fentress placed the parking structure along I-25 (at ground level) to keep it out of the high water table. They designed the

100-ft long roof trusses (l.), erected in three sections, serve as transfer beam, support floor below. Mid section of 100-ft truss (below). Hanger splice plates project below bottom chord (1st vertical from ends).

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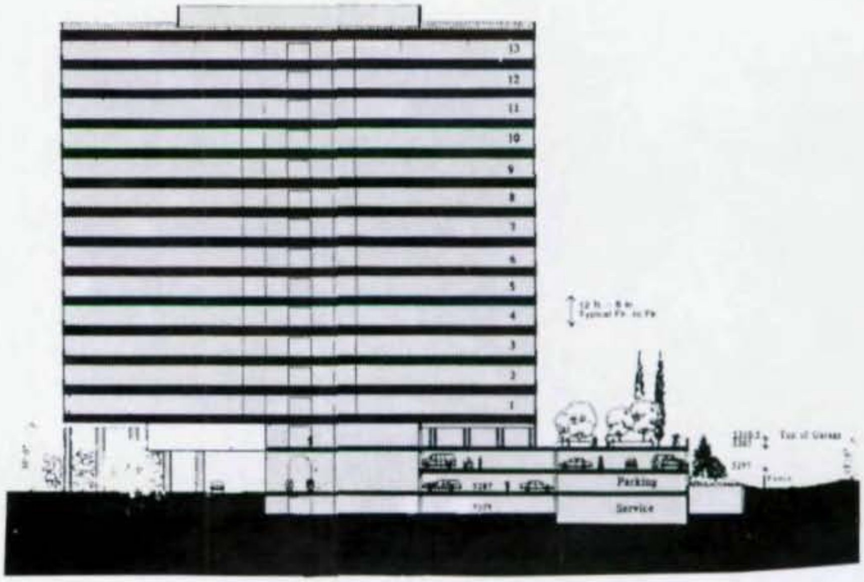
building to go over the parking facility. This eliminated traffic noise problems and provided the site with 50% of open space for a plaza area to permit interaction between the building and its environment.

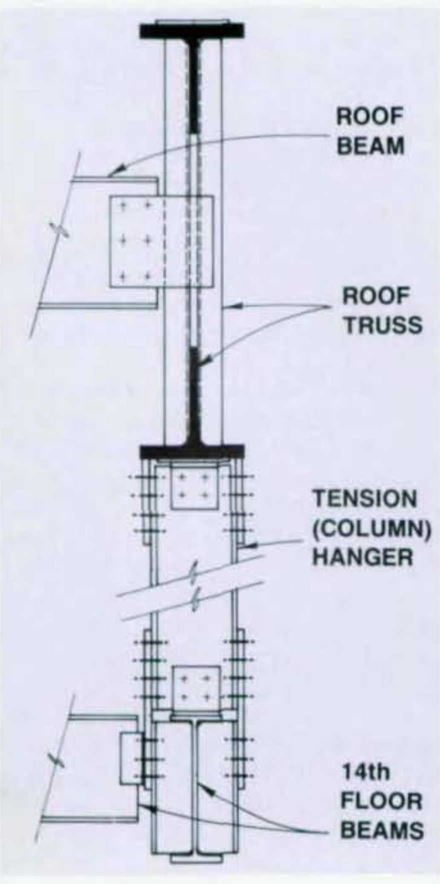
Architecture Possible with Steel

Structural engineer Richard Weingardt Consultants made the architectural design possible by using structural steel instead of concrete. The result—a building frame that preserved the structural integrity of One DTC and carried through its intended feeling of permanence and prestige. More importantly, however, the structural steel frame proved economically viable, as well as providing for the most expedient project schedule.

The structural steel frame, which gave the architect more flexibility in the design process, was also the most logical material for this project. That flexibility played an important part in the success of One DTC and in the ease of its leasability. Using unbraced, 40-ft tall steel columns to lift the building over its parking garage provided both a grand entrance and a drop-off point underneath the elevation. Having elevated the building above plaza level, narrow, full-



Cross section of One Denver Tech Center



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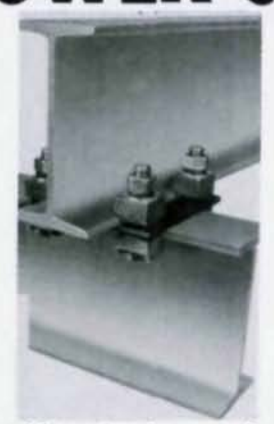

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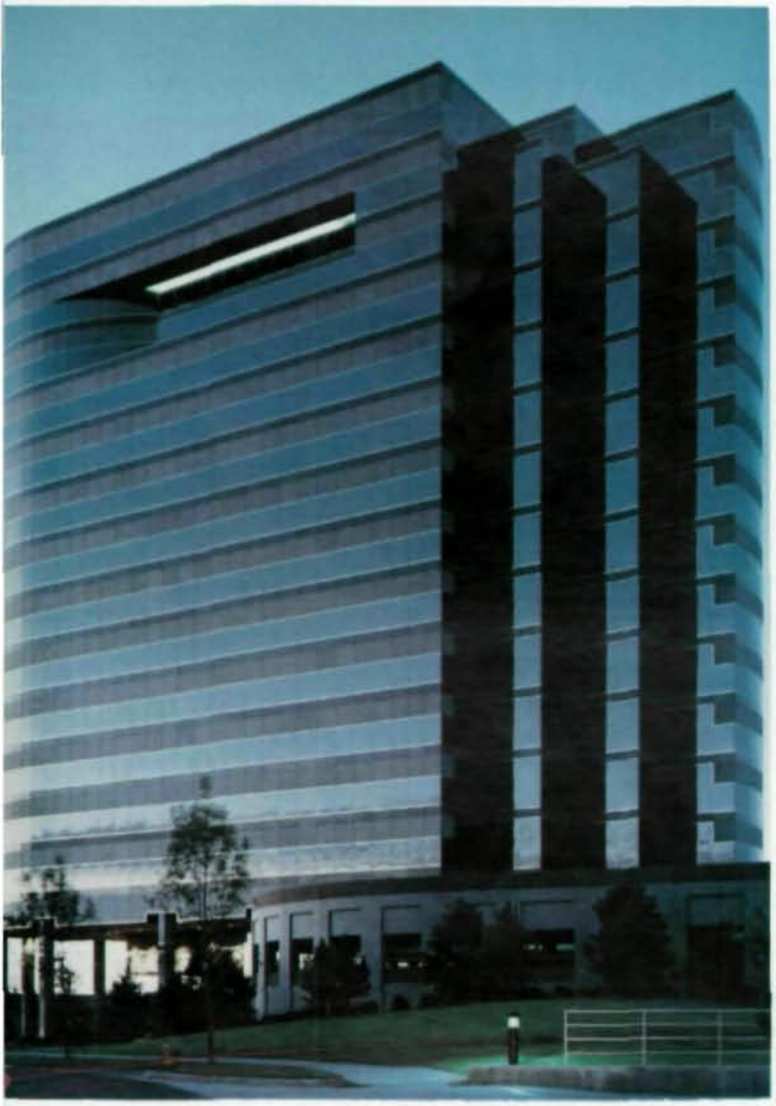
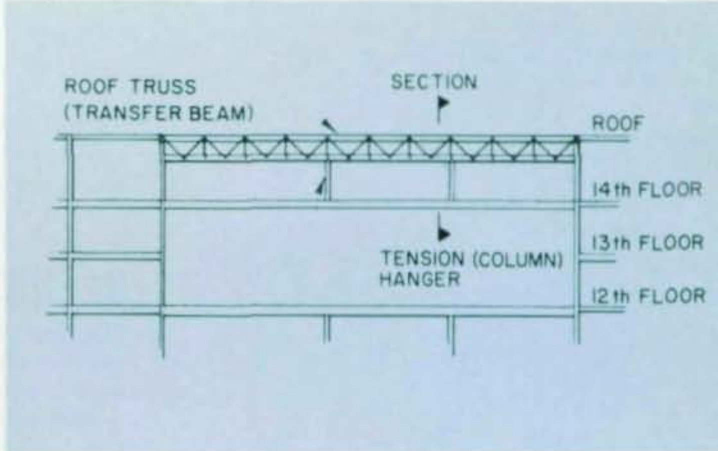
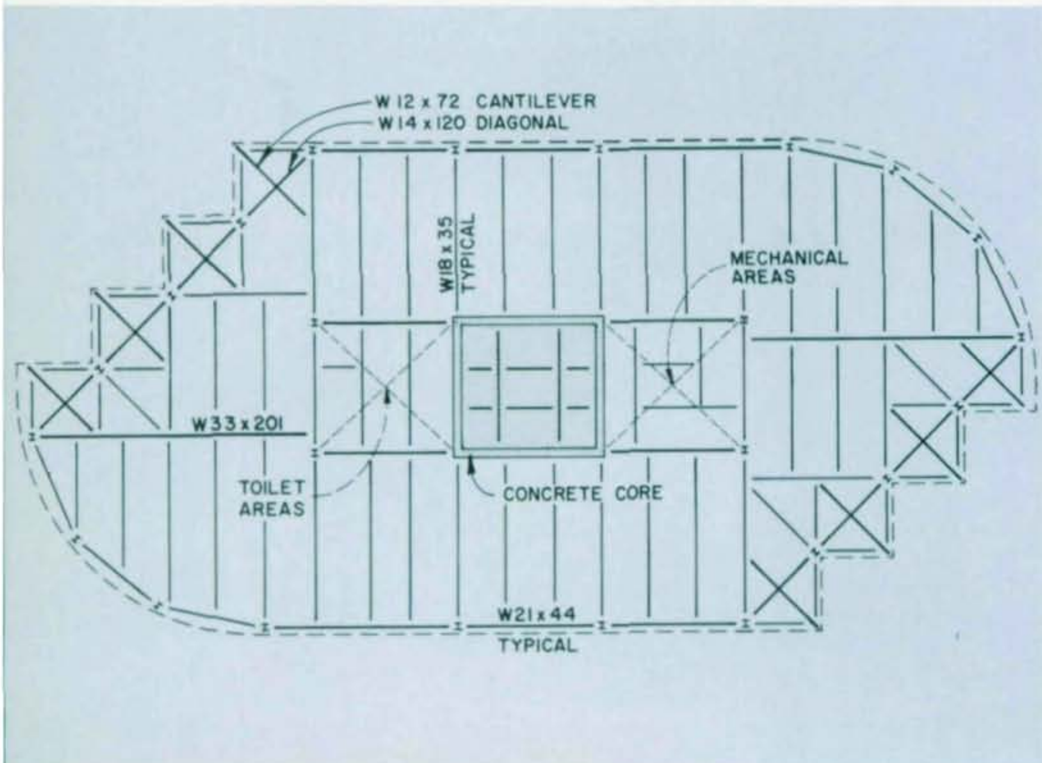




Orientation and elevation of building, layout of parking deck, grand lobby and views create design above usual "office park." Long, cantilevered beams form serrated building ends. Photo by Martin Cole.

height steel tubes were used to support the full-height glass walls against lateral forces.

The greatest advantage in using the structural steel framing system was that it permitted the two distinctive design features of the building. The serrated building ends, the first such feature, were intended to maximize the panoramic views of the building's location. Use of a structural steel frame permitted cantilevers to eliminate columns at the exterior corners of the building. In this way, the oval building designed with cutaways on two diagonally opposing sides, and the cutaway's serrated edges, created column-free corner offices with two-sided exterior views, with the contoured sides providing wide-angle views. As a result, 90% of the occupants boast a spectacular view of the Rocky Mountains as well as downtown Denver.



100-ft Truss Transfer Beam

The second distinctive architectural design feature is the dramatic setback in the building on two sides to create balconies over 150 ft in the air. This two-story "notch" at the 12th and 13th levels truly distinguishes and identifies the building from long distances in both directions. This feature presented a major challenge to the structural engineer—how to support the perimeter of the 14th floor and the roof because the 13th level was setback from the floors above and below. Exterior columns were discontinued at the two-story "notches." The solution was a 100-ft long, deep "transfer-beam" at the roof. Obviously, there was not enough depth in the plenum space between the floor or roof and the ceiling to allow for a girder which could span 100 ft. The solution was to install a single member at the roof level where the parapet depth could be used in addition to the plenum depth. Both trusses and built-up plate girders were analyzed, with a 6-ft deep steel truss selected as the most economical design solution.

No Interior Columns

The typical office floor framing system is steel composite with concrete topping to create long-span, column-free interior space. Beam spans are typically 40-ft and 59-ft, eliminating the need for any interior columns. Typical floor beam sizes are W18 × 35 for interior spans and W21 × 44 for spandrel beams. The 59-ft long interior girders are W33 × 201. To keep the total depth of the cantilevered corner beam over the diagonal support girder at a minimum, a W14 × 120 was used for the diagonal and a W12 × 72 for the cantilever for a total depth of 26 in. Average column sizes are W14 × 132 with W14 × 257's at the building base. The floor plan has 15,000 sf typically, with no interior columns. The first structural floor is 34 ft above ground level, and all other floor-to-floor heights are 12 ft-6 in.

The use of steel permitted the mechanical system to penetrate beams at critical locations so ceiling heights could be maximized. For economy, all structural beams and columns are ASTM Gr. 50. Lateral loads, seismic Zone one and wind forces are resisted by a concrete shear wall core composite with a vertical, structural steel truss framework so the interior core area could be kept as small as possible to provide maximum leasible space.

Special Considerations

Because of its size, forming and erecting the three-piece steel truss was an unusual procedure. One end piece was lifted and joined to the building before erection of the center section, which was then bolted to that end piece. Next, the remaining end section was installed. Erection cranes remained in place to hold the center section and last end section until final connections were completed. Each truss (one on each side of the building) was 100 ft long and weighed 12 tons. Each joint was welded together only after all three pieces were in place.

Structural work was completed at a pace of two floors per week, including work done during the winter months—to complete the building in 12 months.

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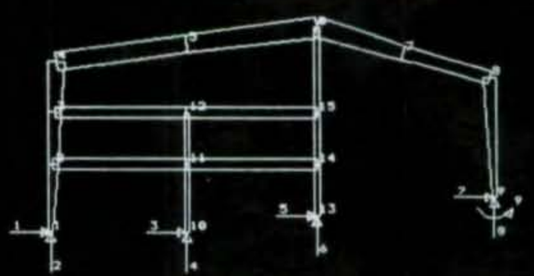

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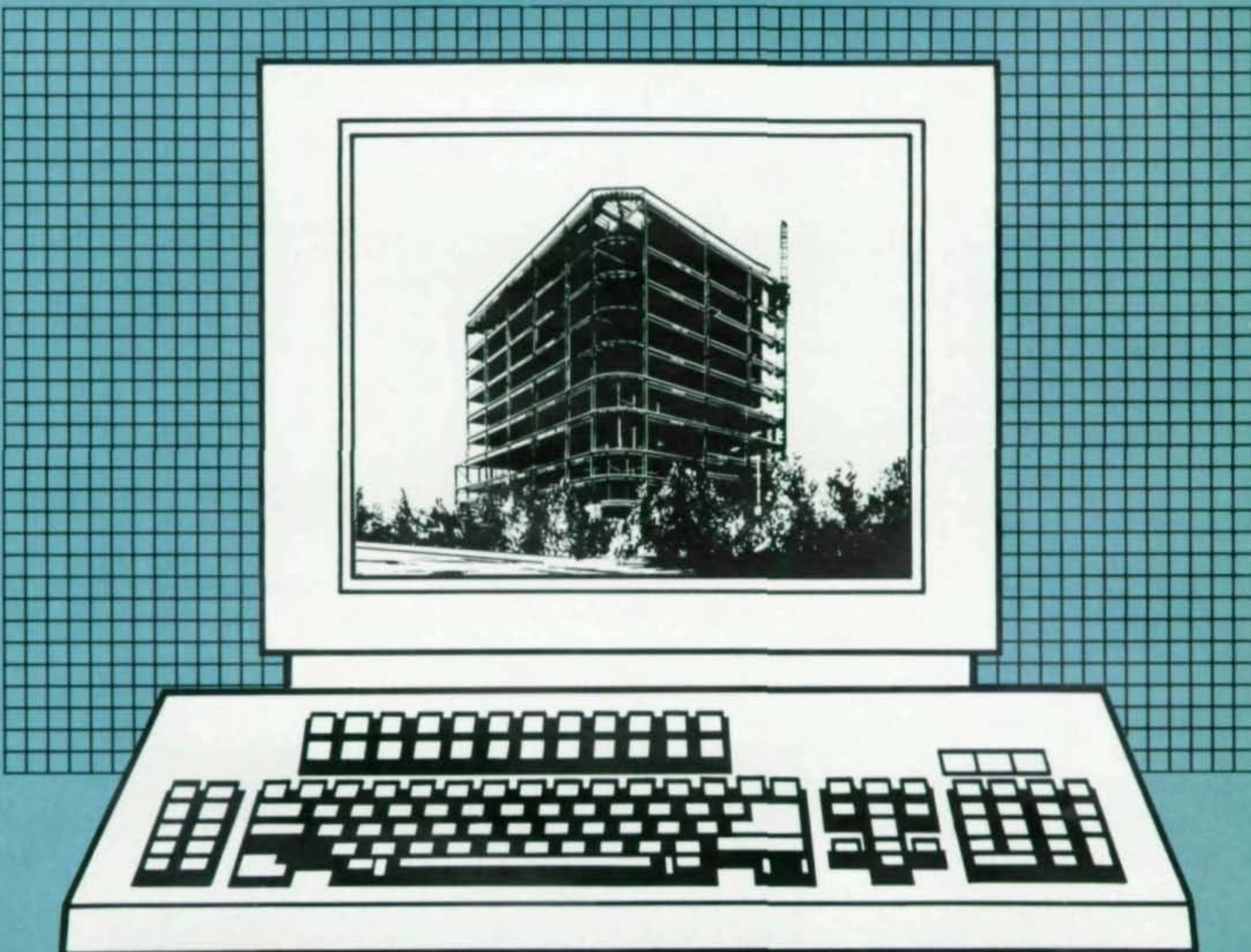

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Steel Notes

ICBO APPROVES LRFD AS NEW UBC STANDARD

The International Conference of Building Officials (ICBO) approved Load and Resistance Factor Design (LRFD) as a new Uniform Building Code (UBC) standard at their September annual meeting in Kansas City, Mo.

LRFD may now be used in seismic zones 2, 3 and 4 when approved by the building official in the design of seismic-resisting frames. However, the design must yield results equivalent to those obtained by use of allowable stress design. This limitation is expected to be relaxed once the new AISC LRFD seismic appendix is submitted to ICBO.

At the same meeting, the ICBO membership also approved a new high-strength bolt installation standard based on the 1985 Research Council on Structural Connections installation standard for A325 and A490 bolts.

The new UBC code and standards will be available in 1988. This model building code is widely accepted in the western U.S.

LIGHTWEIGHT MANUAL AVAILABLE AGAIN

A limited number of the *Manual of Steel Construction*, 8th Ed.—Lightweight (M012) has been reprinted and is now available. At 1 lb.-5¼ oz., 7/8-in. thick, this lightweight Manual is ideal for the job site and travel. Printed on opaque "bible" paper, it is unabridged and complete in every detail. The 832-pg. Manual still contains the provisions of the November 1978 AISC *Specification for the Design, Fabrication and Erection of Structural Steel for Buildings*. Data is divided into six basic sections: (1) Dimensions and Properties, (2) Beam and Girder Design, (3) Column Design, (4) Connections, (5) Specifications and Codes and (6) Miscellaneous Data and Mathematical Tables. The lightweight Manual is $36 for members; $48 for non-members. To order, send check, money order or Visa/Mastercard information (state type of card, number and expiration date) to AISC Publications Dept., P.O. Box 4588, Chicago, Ill. 60680-4588.

T.R. HIGGINS DIES AT AGE 89

Dr. Theodore R. Higgins, former AISC director of engineering and research, died on September 19, at age 89, in Concord, N.H. Higgins joined the Institute in 1940 as chief engineer and was AISC director of engineering and research from 1945 to 1968.

During his tenure at AISC, he contributed substantially to the advancement of the structural steel industry through his innovative engineering, technical papers and professional lectures. He was involved with all aspects of the Institute's wide variety of research activities. And he played an especially prominent role in the more than 20 years of research which led to the development of plastic design techniques for steel structures. He authored the first AISC manual on plastic design, titled *Plastic Design in Steel*.

Higgins was active in the Institute's Committees on Specifications, Code of Standard Practice, Manuals and Textbooks and Steel Structures Research. With the cooperation of the researchers and his staff, he formulated the AISC *Specification for the Design, Fabrication and Erection of Structural Steel for Buildings* and *Code of Standard Practice for Steel Buildings and Bridges*. Under his guidance the AISC *Manual of Steel Construction* and various reference texts and manuals such as the *Design Manual for Orthotropic Steel Plate Deck Bridges, Guide for the Analysis of Guy and Stiffleg Derricks* and the *Structural Shop Drafting Textbook* were published. The *Structural Shop Drafting Textbook* involved coordinating the previous varying practices and viewpoints of the industry into a useful reference and text of the current, more standardized practice. The textbook is currently in its 4th edition.

In 1972, the T.R. Higgins Lectureship Award program was established by AISC to recognize his contributions. The award is given on an annual basis to the author of an article considered to be an outstanding contribution to engineering literature on fabricated structural steel.

Those organizations or individuals wishing to contribute to the T.R. Higgins Memorial-Lehigh University Fund, should contact Lynn S. Beedle, High Rise Institute, Lehigh University #13, Bethlehem, Pa. 18015-3191.

IT'S TIME FOR NEC/COP EXHIBITORS TO BOOK A BOOTH

Organizers of the 1988 NEC/COP National Steel Construction Conference, to be held June 8-11, 1988 at the Hilton Fontainebleau Hotel in Miami Beach, Fla., are now accepting reservations from those who wish to exhibit products and services at the conference. Brochures have been sent to nearly 3,000 potential exhibitors, as identified by AISC. Most of the exhibitors who attended last year's combined NEC/COP conference in New Orleans will be back again this year for the nation's only all-structural steel show.

For the first time, *Modern Steel Construction*, will publish its May/June 1988 issue as the 1988 NEC/COP Official Program Issue. As a special exhibitor bonus, each NEC/COP exhibitor will be entitled to a free 2-in. program ad in that issue. Plus, each exhibitor will be entitled to a 25% discount on any ad one-half page or larger running in the Official Program Issue. (No other discounts apply.) For details on advertising in this special Program Issue, contact *Modern Steel Construction*'s advertising representative, Kirby Palait, at The Pattis Group, Lincolnwood, 312/679-1100.

For further information on attending or exhibiting at the 1988 NEC/COP National Steel Construction Conference, contact Lona Babbington, AISC director of public relations, 312/670-5432.

Fast-Track Construction

by Herbert R. Fletcher

When I went to work for a major architect/engineering firm in Detroit, the bulk of our design commissions, with the exception of auto and tractor jobs, were for conventional bid projects. After a lengthy selection process proposals were solicited and submitted, the decision was made and a design commission awarded. Then, there ensued on often lengthy give and take about where the project was to be built, basic size and configuration, horizontal or vertical thrust, frame material and construction schedule, plus a myriad of other things from toilet seats to ground cover. We would transmit these ideas and requirements to the various disciplines and departments and after numerous meetings and discussions, and with the owner's blessing, proceed to create this building on paper. Not until the last creative ideas from the final respondents were received, incorporated and approved by the owner did we finalize the bid documents.

The documents were printed and issued to a prequalified list of general contractors. The generals would then solicit bids from an impressive number of subcontractors. After deep deliberations and consultations with his resources, the contractor submitted his bid. The owner, with the designer's input, evaluated the bids and, consistent with budget and project schedule, presented and conditioned by various factors, would agree the job be awarded to the most responsive contractor. At this point and after all that energy has been expended not one iota of productive work has been directed towards the actual construction of the project at the site!

Fast Track

The auto and tractor projects were handled in a dramatically different manner. Major trades were bid directly to the owner as the design progressed. Site and foundations followed by structural frame, mechanical/HVAC and electrical were bid as separate packages in sequence. As opposed to the conventional all-trades package, the owner, in conjunction with the designer, would carefully screen subcontractors to qualify them as potential bidders. Financial strength, capacity, quality of work, track record and in-house engineering—among others—were factors in selecting the five or six bidders. The bids were received, opened, evaluated and a contract awarded to the firm most responsive.

Price, although heavily weighed, was not the only factor considered. To receive an order, there was no requirement to cut pricing. The result was the potential to perform without cutting corners and still have room to make a profit for your effort. We had no name for this process, but it is now known as fast tracking.

Fast track is an accelerated design and construction process that allows construction to begin before design is complete. Another name for fast track is phased construction. The very nature of the concept has an imperative that structural steel should be the framing material. Fast track, or phased construction, is not just a buzzword. Combining design and construction into an overlapping and ongoing process requires teamwork and encourages communication between the subcontractor, designer and owner. Each phase of construction starts when the design and specifications for that segment of the project are ready. The object is to achieve beneficial occupancy in a significantly shorter time than is possible with traditional bidding methods.

Fast tracking can encompass the whole project or only selected trades. The structural frame is, almost without exception, the cornerstone of a phased construction effort because "everything hangs from the steel." Of course, to achieve this end, site work and foundations are mutually exclusive in the program. There is a need for a team spirit, and communications have to be significantly closer than normally experienced on the traditionally bid project.

Fast track construction may well be the answer to the "do you want it good or do you want it Wednesday" concept advanced by George Kassabaum of Hellmuth, Obata & Kassabaum. Even in this day of reasonable interest rates, the significant costs of financing, labor and material have increased dramatically the cost of time. These costs pose difficult challenges for today's building and reflect even tougher areas for the construction industry. There is a value concept to be addressed and a requirement the construction industry develop solutions and methods responsive to that need. The alternative to the traditional linear construction process is a partnership between owner, architect and the structural steel contractor to complete the frame faster than if it were contracted for on a conventional basis.

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proach, the owner deals directly with the structural steel fabricator. Without the usual fetters of restricted communications and input, the fabricator can use his specialized knowledge, and as a team member he can introduce practical engineering, realistic scheduling and value engineering into the construction equation. The owner still has the deciding role in the project, but the fabricator's input is not watered down or lost in translation.

This concept of management structure creates a team approach to the project—replacing the historical adversary relationship. The designer's concept can be implemented in the most cost-effective manner and the owner has direct communication affording him a "hands-on" relationship and flexibility not found in the traditional approach. The fabricator can effect a more rational, practical approach to the project challenge. We are now in a circumstance where the fabricator can be as competitive as he is when addressing design/build. And, at the same time, he is shaving significant time off of any project schedule. The frequent adversarial situation put in motion by the conventional bid process is diminished significantly.

Early on, at some stage of the project, usually after foundation and steel contracts are awarded, a general contractor is brought on board on the basis of a hard-dollar bid. The trades, which had been on a direct-owner bid or fast tracked, were then assigned to the successful general for coordination and, on occasion, payment. As the concept spread and became known as fast track, or phased construction, a new factor was added to the equation—construction management.

A New Equation

The construction manager has replaced the general contractor on the fast track team to a significant degree. There are two dramatic differences in both when a construction manager comes aboard and how he contracts with the owner in comparison to the general contractor project. The construction manager may be the first contract let by the owner, preceeding the designer in many cases. And the method of contracting is on a fee basis, with most of the construction manager's workforce generally a reimburseable expense.

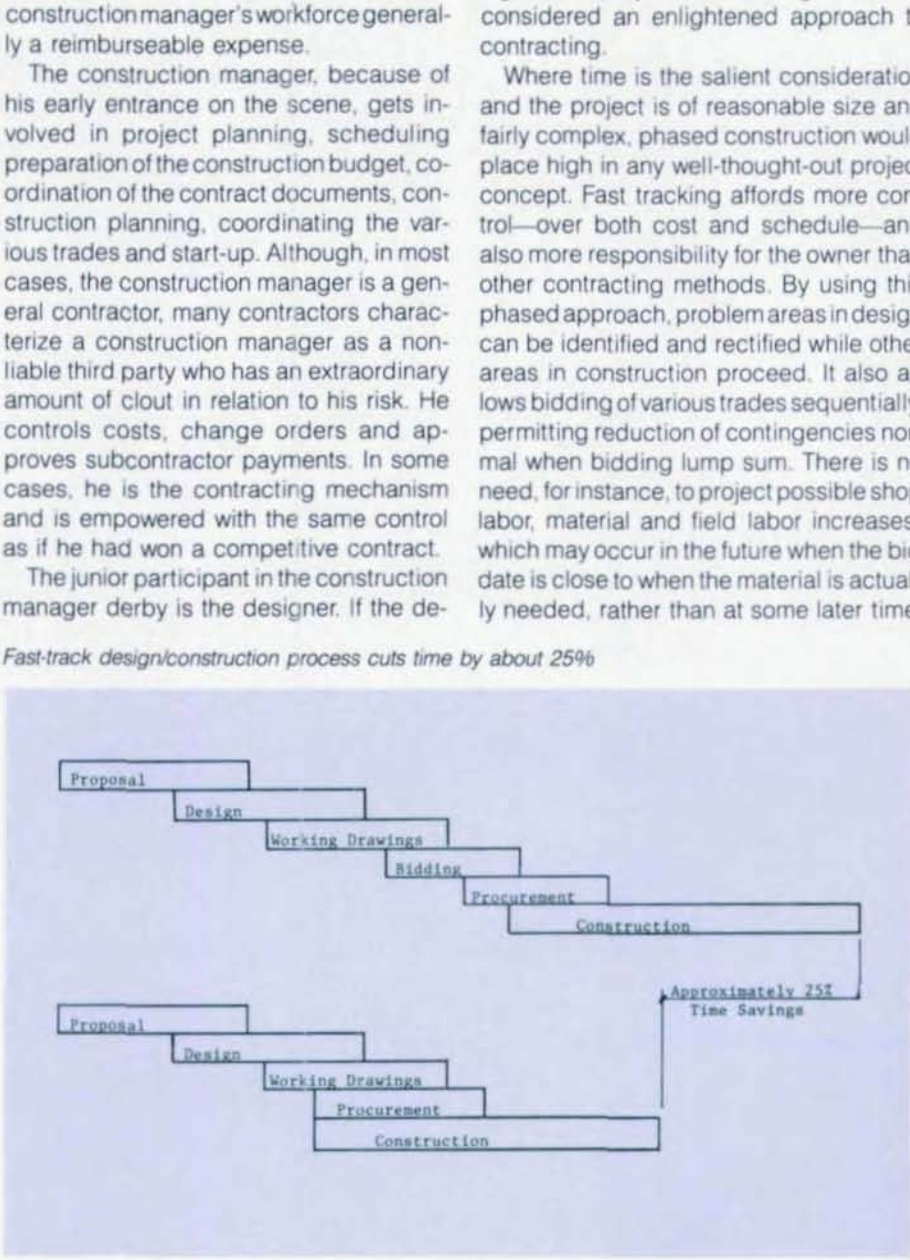
The construction manager, because of his early entrance on the scene, gets involved in project planning, scheduling preparation of the construction budget, coordination of the contract documents, construction planning, coordinating the various trades and start-up. Although, in most cases, the construction manager is a general contractor, many contractors characterize a construction manager as a nonliable third party who has an extraordinary amount of clout in relation to his risk. He controls costs, change orders and approves subcontractor payments. In some cases, he is the contracting mechanism and is empowered with the same control as if he had won a competitive contract.

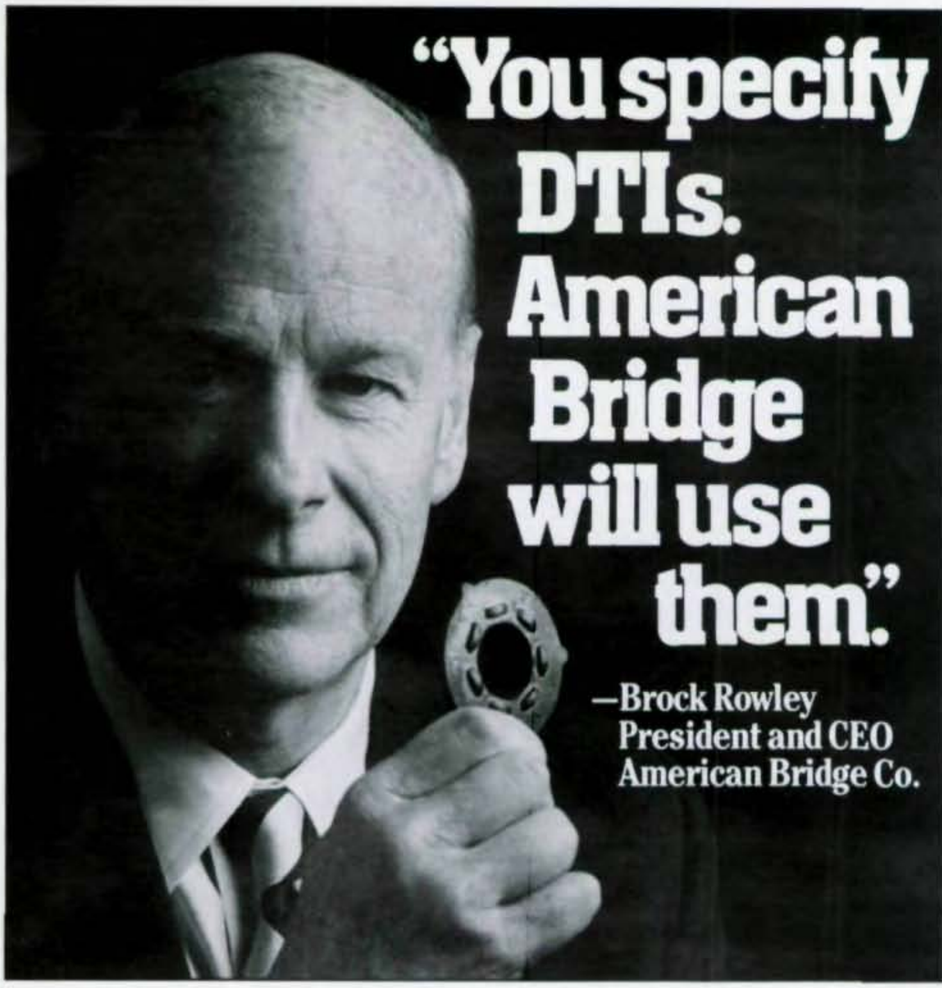
The junior participant in the construction manager derby is the designer. If the designer is also the construction manager he has complete control of the construction process. Although eminently capable to perform the design, and having the experience and resources to do so, there is sometimes a question of his ability and resource to perform at the jobsite. The question I am not qualified to address.

There is also a question as to the validity of the construction manager concept in total, which is certainly not germane in this discussion. Whether a general contractor or construction manager is involved, the distinct trend seems to be away from traditional bidding. And, although there are many other innovative approaches to contracting, fast tracking has been proven to afford savings in both dollars and schedule. We must concede a prequalified group of bidders whose quotations reflect real cost and whose early inclusion result in significant project time savings must be considered an enlightened approach to contracting.

Where time is the salient consideration and the project is of reasonable size and fairly complex, phased construction would place high in any well-thought-out project concept. Fast tracking affords more control—over both cost and schedule—and also more responsibility for the owner than other contracting methods. By using this phased approach, problem areas in design can be identified and rectified while other areas in construction proceed. It also allows bidding of various trades sequentially, permitting reduction of contingencies normal when bidding lump sum. There is no need, for instance, to project possible shop labor, material and field labor increases, which may occur in the future when the bid date is close to when the material is actually needed, rather than at some later time.

Fast-track design/construction process cuts time by about 25%





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Tighter Controls

Fast track construction does then indeed seem to allow the owner to keep tighter control on the project cost and schedule by retaining or delegating responsibility. For the subcontractor, it allows his voice to be heard and, since he is now a prime contractor, gives him more influence over design concept. The potential for value engineering to afford a reward to the firm generating the ideas is multiplied infinitely. The direct payment to the trade contractor cannot be overlooked when evaluating participation in a fast track job. Payment is still 30 days after invoice date, and the construction manager or general contractor must approve the requested percentage completion. But to get direct payment from the owner has to be attractive to the fabricator or any other trade. There is no opportunity for another party, such as a general contractor, to delay temporarily the appropriate distribution of earned payments. There have, as we all know from experience, been instances of non-payment by general contractors stretching significantly beyond the contractual 30 days. Although the trade contractor's job responsibilities do not change as a prime contractor for his defined work, he is connected directly to the owner for payment, or at worst, to the owner through a construction manager. This is certainly a superior position to having to energize a sometimes non-responsive general contractor to present his particular position, problem or need.

Fast tracking gives trade contractors a much more influential role in the construction process. Most construction under a general contractor is, to a great extent, sub-contracted. So contracting directly—at least the major trades—seems to make sense for both owner and contractor. Direct contracting can eliminate the contingency allowance, often a prerequisite of general contract bidding because of the general contractor's proclivity for questioning the value of the subcontractor's bid. More often than not, this contingency money remains with the general contractor and does not ever surface at the owner level—all the more reason to contract directly with major subcontractors.

These foregoing conclusions are my personal opinion and are difficult to prove or disprove. Therefore, I cannot bury you in documentation to support my perceptions. But I do have one comparison of time when evaluating conventional vs. fast track construction in *Business Facilities Magazine*, February 1985. As the bar chart indicates, a time savings of 25% can be attained by fast tracking. The attendant dollar savings is in the 10-15% range over conventional construction. □

Herbert R. Fletcher is vice president of sales for the Haven-Busch Company, Grandville, Michigan.

Structural Considerations in Automated Storage Rack-supported Facilities

by James Lord and Dr. Mostafa Zayed

This paper summarizes the structural considerations in the design and construction of a special type of steel structure—the high-rise, automated storage and retrieval facility. In it, the structural system comprises only light-gauge storage rack components. Storage rack frames are fabricated of small, cold-formed, light-gauge tubes, pipes and channel sections. Seismic design criteria included a requirement to remain essentially elastic during a maximum credible level earthquake ground shaking. Known as "rack-supported buildings," very few such buildings have been constructed in high seismically active regions. The case study outlined here represents one of the first to be constructed in California.

One of first rack-supported structures in California. Aerial (top) indicates size. Interior (c.) down-aisle view of braced-frame system. Long frames (bott.) are laterally stayed by cross-aisle frames.

Introduction

Computer-controlled, random-access automated storage and retrieval systems have proved themselves extremely efficient and cost-effective in high volume, rapid turnover warehousing of pallet-stored merchandise. As a result, dedicated, single-purpose facilities of this kind have proliferated throughout the U.S. in the last decade. The efficiency of these facilities may be measured in several ways, one of which is the ratio of the cube capacity of pallet storage to the volume enclosed within confines of the structure. Another measure relates the cube storage capacity to the floor area involved. In this case study, in which overall building height is about 100 ft and the gross floor area is near 120,000 sq. ft, these two efficiency measures are evaluated as 40% and 40 cu. ft of storage/sq. ft floor area, respectively.

The total pallet storage capacity is 57,150. Pallets are all 4 ft-6 in. x 4 ft in size. A fixed configuration of storage provides for unit pallet loads to vary in height in increments of 3 ft-6 in., 4 ft-6 in., and 6 ft and 6 ft-8 in. The warehouse operation involves the use of stacker cranes to store and retrieve pallets. The stacker cranes are designed to traverse the length of aisles located between storage racks, while simultaneous vertical stacker crane cab movement facilitates access to elevated pallet locations. Operational tolerances and capacity limited the vertical travel motion of the stacker cranes to about 100 ft, thereby governing the overall building height. Area-wise, fire insurance considerations dictated sub-dividing the facility into two approximately equal fire compartments.

Structural System

Historically, pallet rack storage facilities have been designed to be self-supporting, at least for vertical loads, and housed within a structurally separated building shell. On occasion, high pallet storage rack configurations sometimes have received lateral support from the lateral-force resisting system of the building shell, insofar as they are connected to the building shell roof construction. In this case study, there is no building shell structure whatsoever. The pallet storage racks are designed to be totally self-supporting. The exterior roof and sidewall cladding is simply attached to pallet storage framing. Figure 1 illustrates the nature and modular configuration of the 100-ft high pallet storage rack framing. Aisles are located at 24 ft-8 in. o.c. Four post cross-aisle light gauge tubular steel frames are spaced at 4 ft-11 in. o.c. between aisles. The outer 6 ft-8 in. wide bays of the cross-aisle frames are diagonally strut-braced. Pin-ended strut action within the 5-ft wide flue between the cross-aisle frames constrains the lateral motion of the two outer bays to be synchronized and accommodates the fire protection sprinkler feeder piping as well as a full-height, full-length, down-aisle diagonally braced frame of structural steel construction.

The cross-aisle frames directly support 15 pallet load arm and rail assemblies and are horizontally pipe-braced to the centrally located, down-aisle braced frames. Roof construction of light-gauge steel decking welded to structural steel framing acts as a horizontal diaphragm to provide limited distribution of lateral seismic loads from one pallet storage rack module to another in the event of uneven pallet load use. Founded upon compacted fill and natural soils, a reinforced concrete mat footing, varying in thickness from 20 to 36 in., supports the closely spaced post configuration of the pallet-storage racks.

The lateral seismic force-resisting system in the cross-aisle direction comprises the 6 ft-8 in. wide diagonally braced frames. These 100-ft high frames act as vertical trusses cantilevering from the floor slab-on-grade. Pallet seismic loads are transferred directly to these cross-aisle

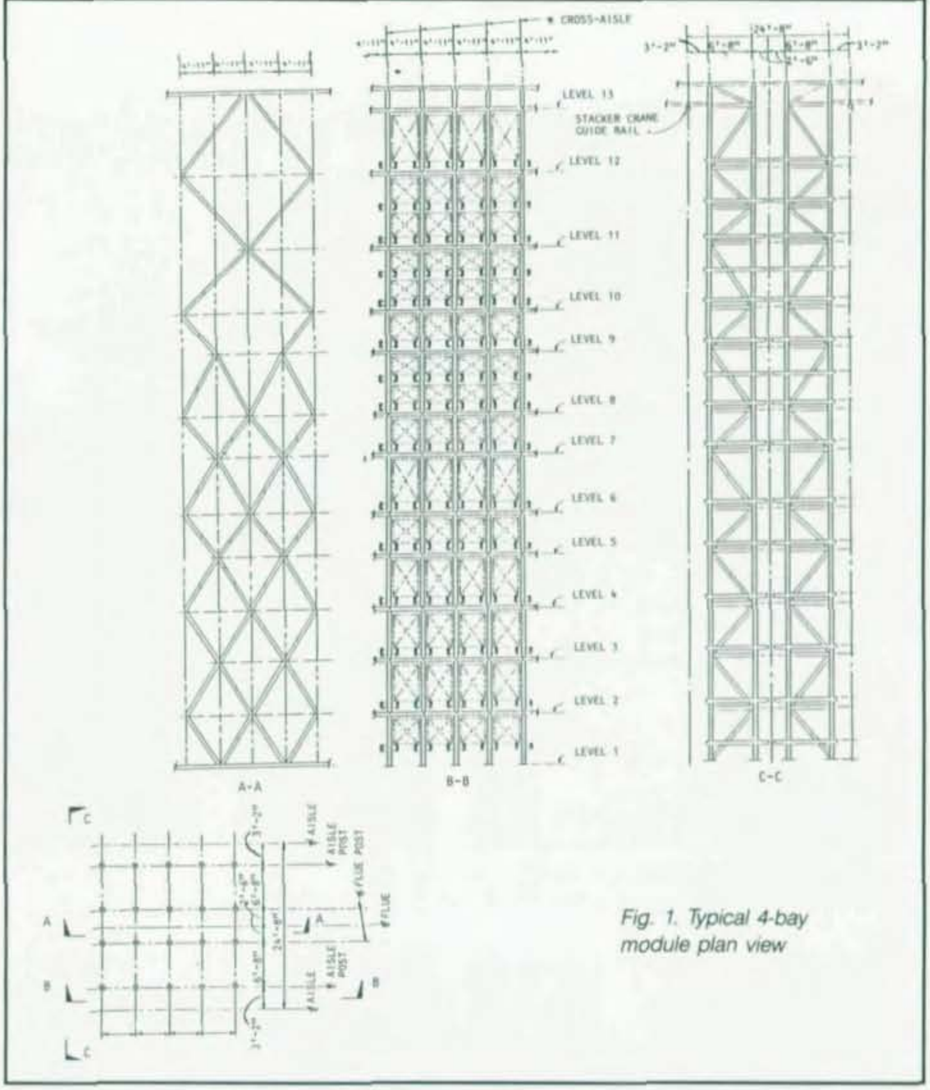
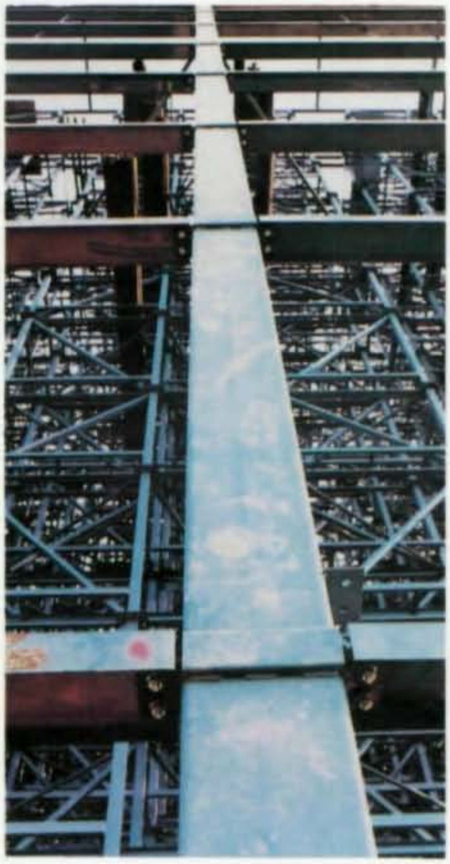


Fig. 1. Typical 4-bay module plan view

Sections of cross-aisle frames are field-welded in on-site welding jig.



Post splice connection detail (l.). Installation of collar plates (r.) at typical aisle and flue post conditions

Collar plate condition at typical flue post (l.). Uplift base plate (r.) at ends of down-aisle frames.

frames through the load arm/rail assemblies. Seismic resistance in the down-aisle direction is provided by the diagonally braced steel frames located at the center of the flue space. These 100-ft high, 200- to 400-ft long frames also cantilever vertically from the floor slab-on-grade and are laterally stayed at vertical intervals by the cross-aisle frames. In addition, they are designed to provide lateral support to a 33 ft-6 in. high low-bay structure attached to the high-bay structure at the east end. Wind loads in the cross-aisle direction are resisted by the single unit cross-aisle frames located at the north and south walls. These frames are designed to span vertically 100 ft from the floor slab-on-grade to the roof diaphragm. Down-aisle oriented wind loads are resisted by the seismic force resisting system.

Seismic Design Criteria

Except for the sidewall cladding cross-aisle frames and the exterior non-structural cladding, the structural design was entirely governed by seismic considerations. The unusual and irregular structural configuration, coupled with a close proximity to California's notorious San Andreas Fault, mandated special seismic design-criteria which recognized the dynamic characteristics and response of the structure during anticipated adverse seismic conditions. Because of limitations in the static analysis seismic provisions in the applicable Uniform Building Code, 1982 edition, and other industry codes and standards normally utilized in the design and fabrication of storage rack structures, the Los Angeles Dept. of Building and Safety required that their dynamic analysis criteria be followed as a minimum. In essence, this criteria prescribes that for braced-frame steel structures, the seismic force-resisting system remain entirely elastic during a maximum credible earthquake. Yielding and buckling capacity limitations must be observed, under the action of simultaneously applied orthogonal components of surface ground motion in which 100% of one horizontal component is combined with 30% of the other horizontal orthogonal component, and vice versa.

Further, vertical earthquake ground motions are also to be considered in conjunction with horizontal earthquake ground motion components. In addition, P-delta effects associated with predicted lateral distortions are to be incorporated, as well as a 1½% overall drift limitation and 7% of critical damping for a maximum credible seismic event. Gravity loading for the MCE design condition included dead loads associated with the storage rack, building enclosure, stacker crane and other equipment and 50% of the rated capacity of the pallet live-load storage. This live-load percentage was derived from a comprehensive pallet utilization survey conducted on a comparable warehouse facility. This survey also led to appropriate recommendations relating to the weight distribution of pallet-stored merchandise.

One very important business interruption-related consideration, not prescribed by the foregoing criteria, pertains to limiting horizontal cross-aisle accelerations in the structure, to those of the earthquake ground motion experienced at the site. This owner-imposed criterion required that time-dependent dynamic seismic analyses be performed using surface ground motion excitation, representative of the various types, durations and intensities of earthquake source events which may affect the site. Source events considered included:

- M8.5 San Andreas event, ~ 25 miles distant, 60 sec. duration
- M7.5 Santa Monica event, ~ 1 mile distant, 30 sec. duration
- M7.5 Newport/Inglewood event, ~ 13 miles distant, 30 sec. duration

To achieve the desired seismic response with regard to mitigating the amplification of cross-aisle horizontal accelerations up through the structure, a limitation of 1¼ to 1¾ seconds was imposed on the fundamental period of lateral vibration (T) in this direction. Further, the use of load rail stopper devices or abrasive, non-skid tape on the top of the load rails was recommended to provide further assurance of preventing sliding of the plywood pallets on the smooth painted finish of the steel load rails.

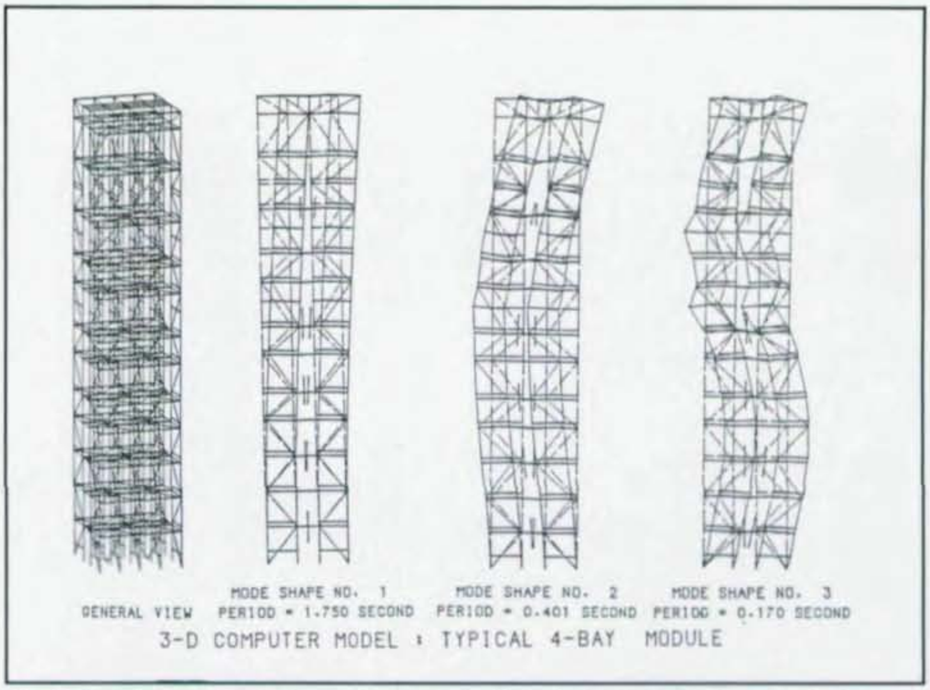


Fig. 2. Typical cross-aisle framing, natural periods of lateral vibration and relative mode shapes.

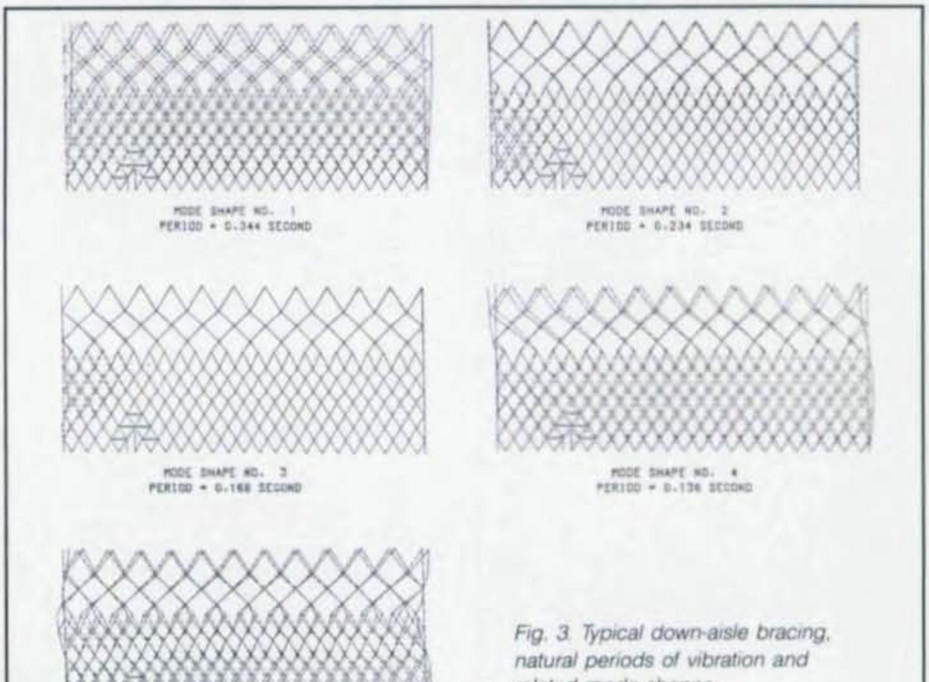


Fig. 3. Typical down-aisle bracing, natural periods of vibration and related mode shapes

Seismic Dynamic Characteristics and Response

The 3-dimensional aspects of the computer modelling was performed on a four bay typical module. All structural members were included in the finite element computer model, which comprised beam-column and truss elements to represent the physical structure. The natural periods and mode shapes of lateral vibration of the cross-aisle frames are shown in Fig. 2. Reference indicates the fundamental period (T) is 1.75 seconds, a value at the upper bound of that specified to control lateral accelerations within acceptable limits. The MCE linear elastic dynamic response calculation indicated a 12-in. maximum cross-aisle roof displacement, an overall building drift of ~ 1% and a maximum lateral absolute acceleration of ~ 50% of gravity. The MCE base shear coefficient approximated 30%. Overturning-wise, the maximum MCE uplift at the typical post condition was computed as 63 kips at the post baseplate.

The dynamic characteristics of the typical down-aisle braced frame system are shown in Fig. 3. In this direction, the fundamental period of lateral vibration (T) was computed as 0.34 seconds. This resulted in an MCE design base shear coefficient of 0.75, a maximum drift coefficient of 0.17% and a 2-in. roof deflection. Significant uplift forces associated with MCE overturning, resulted in the use of heavy embedment baseplates at each end of the down-aisle frames.

Fabrication and Installation

To achieve the desired structural and seismic performance goals and remain within the normal fabrication and cost picture associated with light gauge steel structures of this type, an innovative concept in joint design had to be developed and implemented. This involved the use of a horizontal "collar" plate to connect the many tube, pipe, angle and channel members. Fillet shop welding of the 4-in. square tube posts and the 3/16-in. thick collar plates, allow the thin-walled posts to maintain their structural integrity at vital connection points. Further, the collar plate serves to transmit seismic forces around the light-gauge posts, thereby facilitating use of 8- to 14-ga. thick walls in the tube posts.

Shop fabrication of the cross-aisle frames in 50-ft high section was necessary to satisfy extremely tight tolerances prescribed by operational constraints of the aisle stacker cranes. The 50-ft sections of the cross-aisle frames are field-welded in an on-site welding jig. Figure 4 shows the post splice connection detail used to develop the full tensile capacity of the post. This double-vee type of connection splice avoids heavy stress concentrations at a single plane. This is particularly desirable in welding light-gauge sections, when their effectiveness is vital in maintaining stability during adverse seismic conditions. Tight erection tolerances also require the use of a separate post baseplate and various size shim plates. Difficulties in the field placement of conventional anchor bolts, mandated the use of high tensile "Super-Kwik" bolts and a baseplate template. This in turn, required accurate placement of top steel reinforcement in the floor slab. Only very minor re-bar interference problems were experienced during the course of construction.

Typically, horizontal braces are of 2 to 2½-in. dia. light-gauge pipe construction. Of particular concern was the use of the commonly used, but economical, flattened end condition to effect a single shear connection. The double-lip, flattened pipe condition proved satisfactory in a pre-fabrication tension and compression strength test, although this end condition does govern the ultimate strength capacity of the pipe.

Down-aisle, braced-frame members are all of field-bolted construction. Individual members are bolted in place. Insofar as the 100-ft high cross-aisle frames are unstable prior to the installation of these down-aisle frame members, temporary down-aisle oriented erection guys were needed initially to provide the necessary staying.

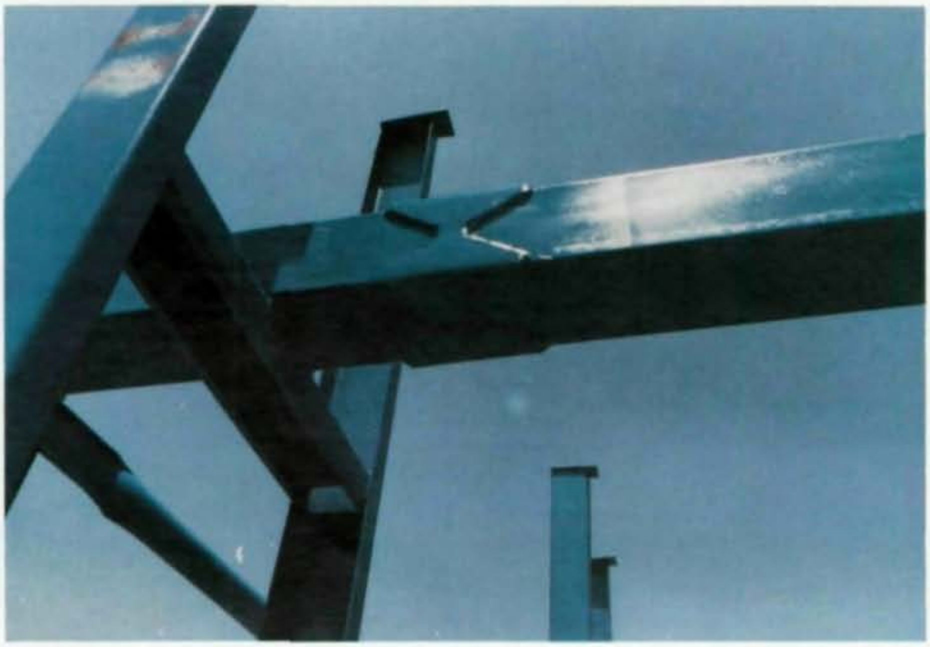


Fig. 4 (top). Post slide connection detail used to develop full tensile capacity of post. Typical pipe strut end condition (bott.).

Conclusion

The case study described indicates the construction of high-bay, automated storage rack-supported buildings of this type is feasible and cost-effective, even at highly seismically active site locations. In this case, a 100-ft high pallet-rack storage facility extending over 120,000 sq. ft and accommodating ~ 57,150 pallets has been designed to withstand anticipated, but adverse seismic conditions with little or no construction cost premium over one designed in accordance with the minimum seismic provisions set forth in most current building and industry codes. Construction entailed fabrication of 900 tons of structural steel at a cost of slightly less than $0.90/lb., inclusive of installation. □

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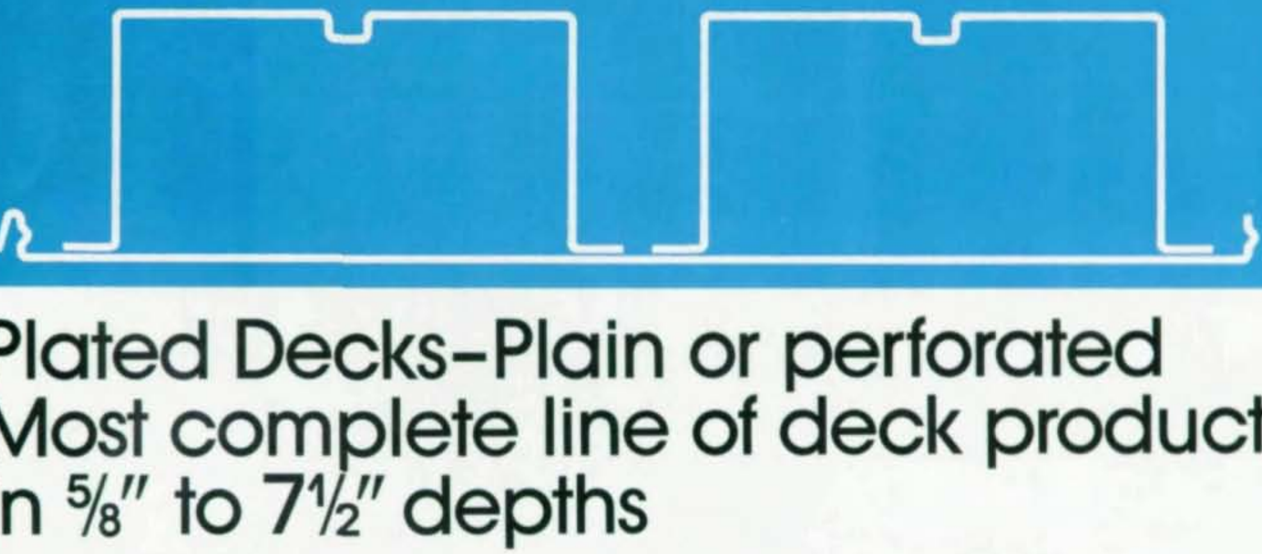

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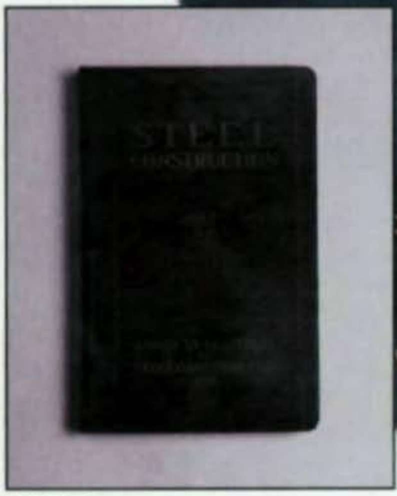

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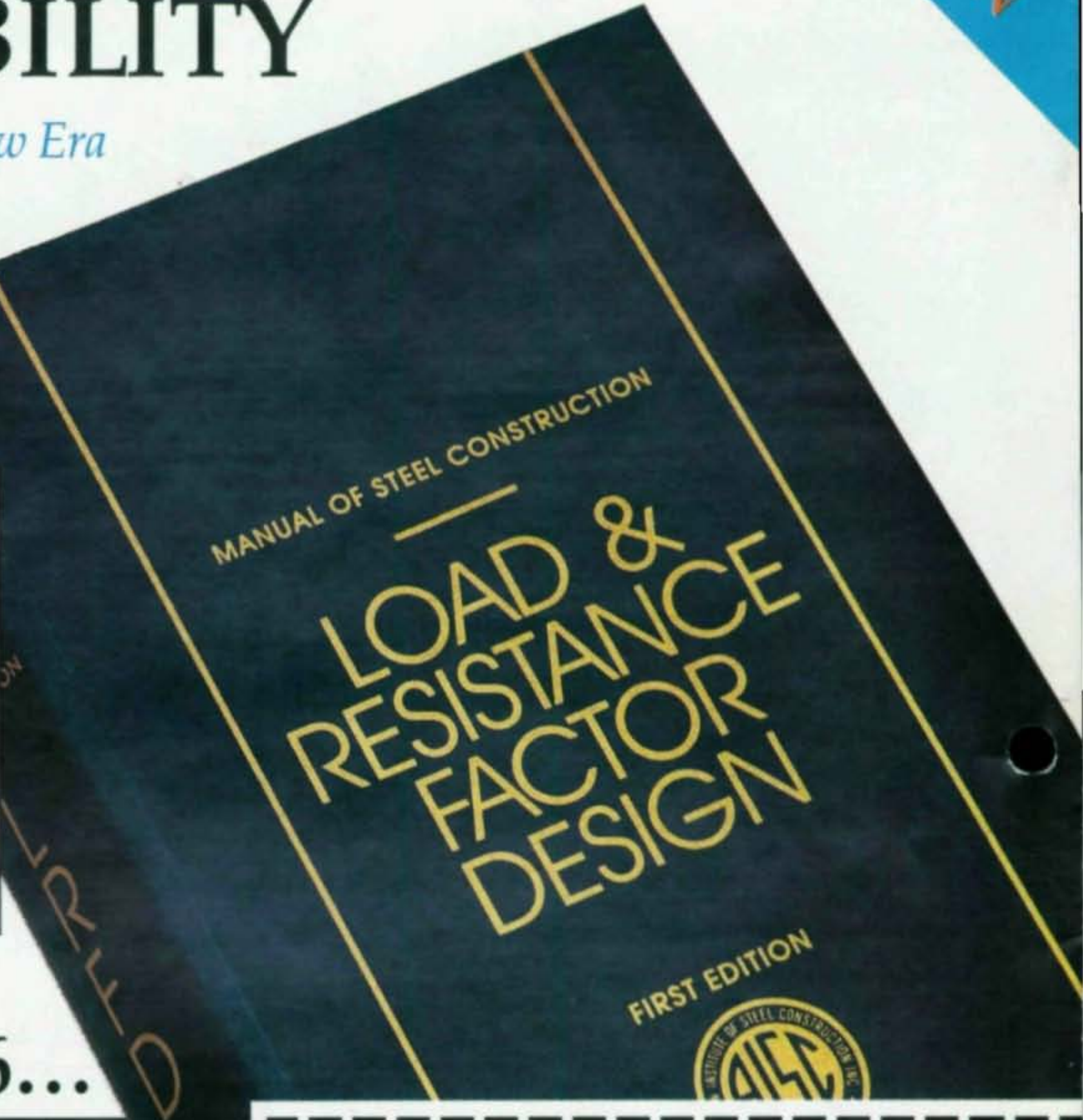

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