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

MODERN STEEL CONSTRUCTION

FOURTH QUARTER • 1986

THIS ISSUE

Design Solutions with Staggered Truss
Long-span Bridges Solve Access Problems
Fire-protection Aids
Superhub—Under Budget, Ahead of Schedule
Why LRFD?
1986 AISC Prize Bridge Winners

DECK DESIGN DATA SHEET

NO. 8

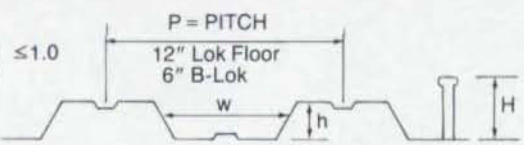
COMPOSITE BEAM DESIGN INFORMATION

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

DESIGN VALUES FOR STUD SHEAR CONNECTORS (KIPS)

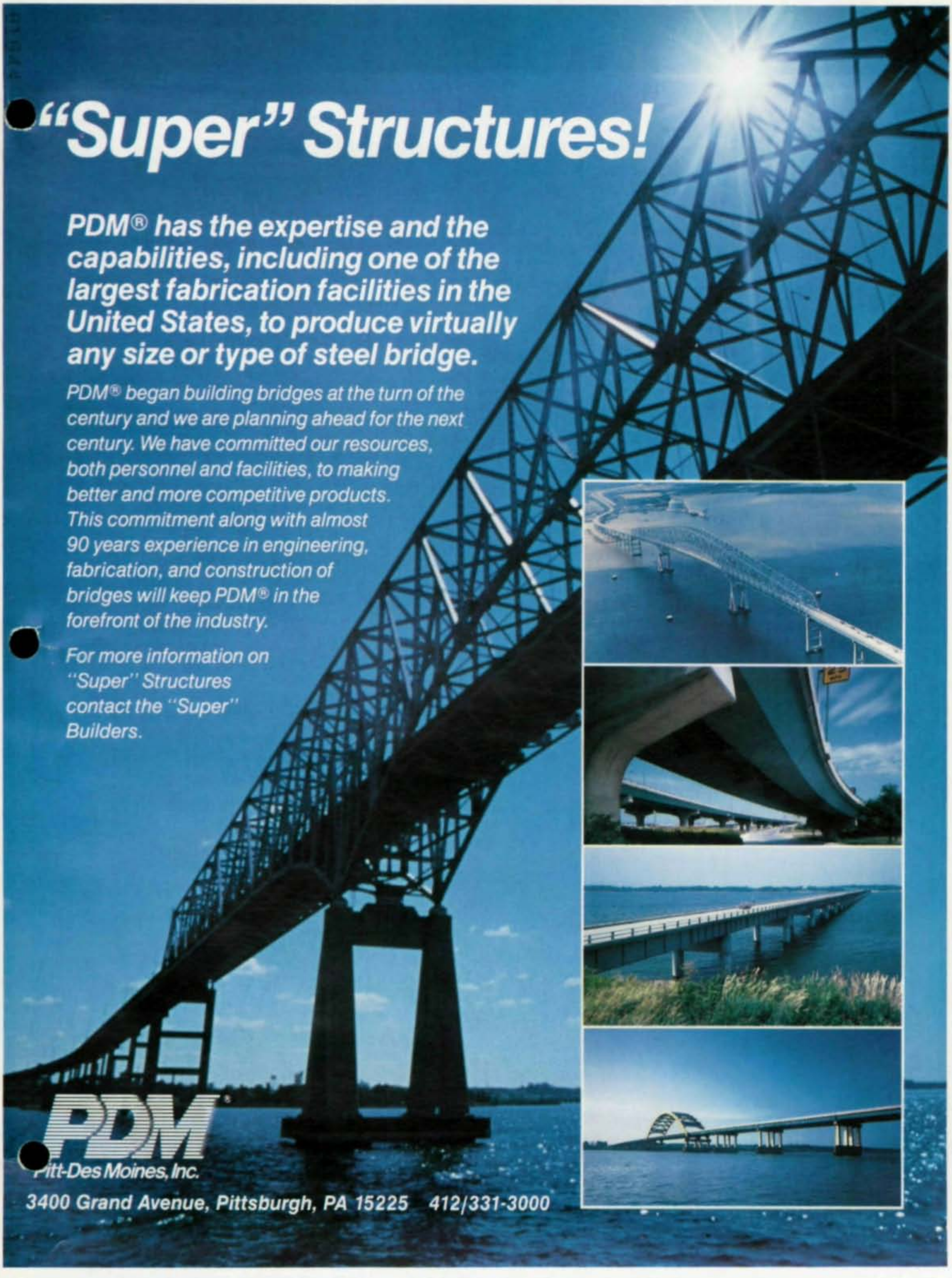
UNITED STEEL DECK, INC. SLAB TYPE			¾"Ø STUD SIZE	w/h OF DECK	NUMBER OF STUDS PER RIB	RIB COEFFICIENT	ASTM C33 NORMAL (150 PCF) WEIGHT CONCRETE			ASTM C330 LIGHTWEIGHT (115 PCF) CONCRETE		
							3000 PSI	3500 PSI	4000 PSI	3000 PSI	3500 PSI	4000 PSI
SOLID CONC.			3"	—	—	1.0	11.5	12.5	13.3	9.9	10.8	11.4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

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



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

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


"Super" Structures!
PDM® has the expertise and the capabilities, including one of the largest fabrication facilities in the United States, to produce virtually any size or type of steel bridge.
PDM® began building bridges at the turn of the century and we are planning ahead for the next century. We have committed our resources, both personnel and facilities, to making better and more competitive products. This commitment along with almost 90 years experience in engineering, fabrication, and construction of bridges will keep PDM® in the forefront of the industry.
For more information on "Super" Structures contact the "Super" Builders.
PDM
Pitt-Des Moines, Inc.
3400 Grand Avenue, Pittsburgh, PA 15225 412/331-3000

MODERN STEEL CONSTRUCTION

VOLUME XXVI • NUMBER 4
FOURTH QUARTER 1986

American Institute of Steel Construction, Inc.
The Wrigley Building
400 North Michigan Avenue
Chicago, Illinois 60611-4185

5 Resorts International
Design Solutions with
Staggered-truss System

14 Steel Notes

15 World Financial Center
Long-span Pedestrian Bridges
Solve Access Problems

18 Design Aids for
Fire Protection

25 Purolator Courier's Superhub
Under Budget, Ahead of
Schedule

30 Why LRFD?

32 1986 AISC Bridge Awards

RESORTS INTERNATIONAL
Design Solutions with Staggered Truss System

by Michael P. Cohen

During the conceptual design stages of most building projects, the structural engineer must consider many different factors before selecting the final structural system. While some basic building properties such as height, shape, usage, etc. lead the structural designer toward certain proven systems, it is not infrequent that architectural constraints, owner requirements and/or building location render these systems unacceptable. Other factors which enter the selection process include: local economic conditions (both of materials and labor), construction schedule, design loads (vertical and lateral), building behavior and occupant comfort, foundation considerations and coordination with mechanical systems. While these general considerations are required on all projects, each building usually presents the designer with an additional set of its own unique problems.

This article is an outline of the conceptual design and selection process by which the staggered-truss system of structural framing was selected for one particular project. The subsequent design process, encountered problems and their associated resolutions, are also reviewed.

Conceptual Design and System Selection

At the start of design, the following parameters were immediately defined: The building was to be a high-rise, luxury hotel located on the oceanfront in Atlantic City, N.J. Designed around a double-loaded center corridor, width would be about 70 ft. An Atlantic City zoning ordinance limited the height of the building to 420 ft. The lower four floors would be public spaces, elevations of which would coincide with those of an adjacent low-rise convention facility around the base of the tower. The remaining upper floors would be guest rooms, with the total number required by the owner about 1,200. Given the luxury hotel status with 24-hour occupancy and the oceanfront wind exposure, occupant comfort (i.e., building drift and acceleration) was of prime concern from the beginning.

Geotechnical investigation at the site found a subsoil of three distinct sand layers separated by two relatively thin clay layers. The project soils engineers determined that under anticipated column loads in excess of 11,000 kips, the clay layers would experience excessive consolidation, resulting in unacceptable building settlement. The foundation selected consisted of deep piling penetrating through the clay layers and bearing on the deepest, dense sand layer located about 90 ft below grade.

Minimum design loads for the structure were governed by the BOCA *Basic National Building Code* which is New Jersey's building code. Typical live loads of 40 psf for guest rooms and 100 psf for corridors and lobbies were applied. Live loads in the public spaces at the lower levels were, however, increased to 150 psf to account for special uses and large assemblies possible at a convention and casino facility of this type. Question was raised as to whether lateral wind loads, as outlined by BOCA, would have to be modified to account for the unusually exposed location in an open coastal region. The design team (owner, architect and engineer) decided on wind-tunnel testing to confirm lateral design loads, measure

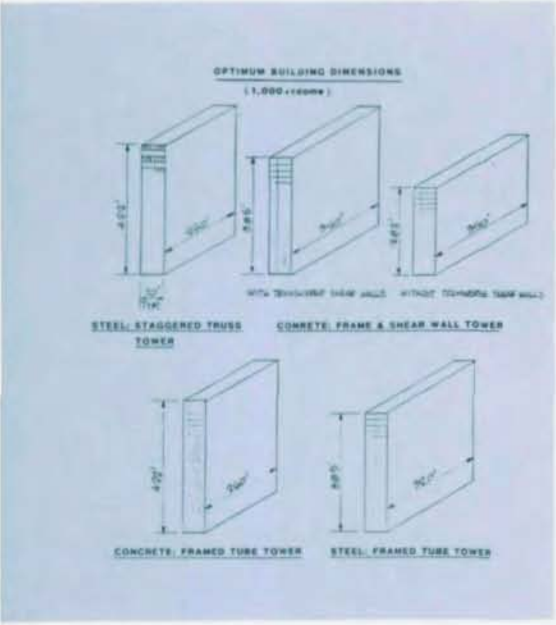


Figure 1

building response, determine cladding pressures and study effects on pedestrian traffic at the building base.

A preliminary design analysis and subsequent cost study was made of four basic structural systems. Starting with 1,000+ rooms as a requirement, building dimensions of each of the systems varied in order to optimize each structural system. The systems considered were (Fig. 1):

1. Steel: Staggered truss. A basic steel building frame with concrete floor system.
2. Concrete: Frame and shear wall. This system consisted of a combination of shear walls and frames to resist lateral wind loads. Two schemes of this type were evaluated, one with shear walls at the central elevator core only and the other with additional transverse shear walls.
3. Concrete: Framed tube. This system utilized rigid concrete frames with closely spaced columns in combination with shear walls at the central elevator core.
4. Steel: Framed Tube. This system was based on rigid steel frames with closely spaced columns in combination with shear walls at the central elevator core.

The preliminary analysis confirmed the structure's relatively high height/width ratio of approximately 6.0 controlled the design. All of the schemes, except the staggered trúss, required relatively large shear walls to control drift. These heavy walls in the foundation proved to be expensive and have negative effect on the architecture of the building. All concrete schemes had substantially higher foundation costs, were more labor intensive and had longer construction time. Resistance to overturning and the resulting uplift force was a problem for the steel-framed tube scheme. The steel staggered truss scheme had none of these handicaps. The relative structural unit costs per square foot were as follows:

1. Steel-staggered truss - 1.0
2. Concrete frame to shear wall - 1.2
3. Concrete-framed tube - 1.10
4. Steel-framed tube - 1.40

From this study, it was apparent the staggered-truss system was the most economical choice for this project. Construction time for the system was expected to be the fastest, and the foundations the smallest of the group. Fast erection is inherent in the staggered-truss system. Architectural benefits included elimination of all shear walls and the large column-free spaces provided at public levels. Considering all factors, the decision was to proceed with the design of the staggered-steel truss tower.

The Staggered-truss System

The staggered-truss framing system was developed by a U.S. Steel-sponsored research team working at Massachusetts Institute of Technology in the mid 60s. The object of the study was to arrive at a new, efficient, structural steel system which would also provide architectural benefits of open interior space and minimum floor-to-floor heights. The result was the staggered-truss system which has since typically been used in a variety of mid-rise (15 to 20 stories) hotel/motel and housing structures. While buildings of great heights were envisioned in the original study, only recently has the technology been developed to permit the design and construction of staggered-truss high-rise buildings greater than 30 stories.

The basic element of the staggered-truss system is the story-deep truss which spans the full width of the building located on alternate floors at each column line. The trusses are supported only at their ends to exterior columns and are arranged in a staggered pattern on alternate column lines. There are no interior columns. The gravity loads are delivered to the truss from the floor slab system which spans from the top chord of one truss to the bottom chord of the adjacent truss. Therefore, each truss is loaded at its top and bottom chord and the total gravity load of the building is transferred to the building's exterior columns on the long side of the building.

The main structural benefit and subsequent efficiency is the system's ability to resist lateral loads acting parallel to the trusses. In long, narrow rectangular buildings of this type, lateral resistance in the transverse direction is often a problem, since the wind forces developed on the large face of the building are substantial and must be resisted by the shorter building dimension or weak axis. The inherent benefit of the staggered-truss system is that the entire building weight is mobilized to resist the overturning moment. The large gravity loads on the exterior columns are usually sufficient to counteract uplift due to wind.

Lateral forces are transmitted through the floor, which acts as a diaphragm or "deep beam," to the top chord of an adjacent truss. The force is then transferred down between floors through the truss web members, with the truss acting as a shear wall. Since the load at the lower chord cannot continue straight down because there is no truss immediately below (no stiffness), it must once again be transferred through the floor system to the top chord of the adjacent trusses. In this manner, lateral forces are transferred back and forth all the way down to the lateral resisting system at the building base. Since the trusses which span the full width of the building have been sized to carry two floors

Typical truss is bolted. Note shop-applied channels on bottom chord, shear studs on top for critical shear force transfer between slabs and trusses.

of building gravity loads, their member sizes are often a sufficient size to provide the required lateral stiffness. This fact, along with the one-third increase in allowable stresses, may not require increased member sizes to resist wind forces.

The floor system is a critical element to the proper functioning of the staggered-truss system. The floor must function as a shear diaphragm to resist lateral loads. Trusses carry the cumulative lateral total load from the building above over a two-bay width. The floor area on each side of the truss must transfer half of this load to the top chord of the adjacent truss in the story below. Therefore, the floor system must be designed to provide sufficient in-plane diaphragm strength and stiffness to sustain these horizontal forces as well as gravity loads.

Since the flow of the transverse lateral loads is through the trusses and the floor system, there is no bending induced in the columns in the transverse direction. The drift of the structure in this direction is a function of the slab and truss stiffnesses and the column cross-sectional areas. It is therefore advantageous to orient the columns with their weak axis parallel to the longitudinal direction of the building. This allows the columns (bending in their strong axis) to be attached easily to the spandrel beams, with rigid connections creating portal frames to resist lateral loads in the longitudinal direction.

The staggered-truss system is a relatively new structural concept based on the sound principle of using the same structural elements to resist gravity and lateral loads. It has been proven very efficient. When conditions are proper, it can yield great economy and maximum architectural flexibility.

Design Application

To accommodate the required number of hotel rooms, the overall number of floors and building dimensions were fixed. The plans called for four public floors, 38 guest room floors, an attic space and a high roof level. The length of the building is 350 ft and the width 68 ft, with projections at the elevator core (Fig. 2). The height from grade to the high roof is around 420 ft.

Once the decision was made to proceed with the staggered-truss design, building dimensions were finalized. Preliminary gravity analysis indicated the chord members of the trusses should be large W10 shapes. Combined with an 8-in. floor system, it made a typical floor-to-floor height of 8 ft-10 in.

A wind tunnel study was conducted based on the geometrical information and a preliminary idea of the dynamic properties of the building. Included were effects of local surroundings, including possible future buildings, these were modeled and monitored

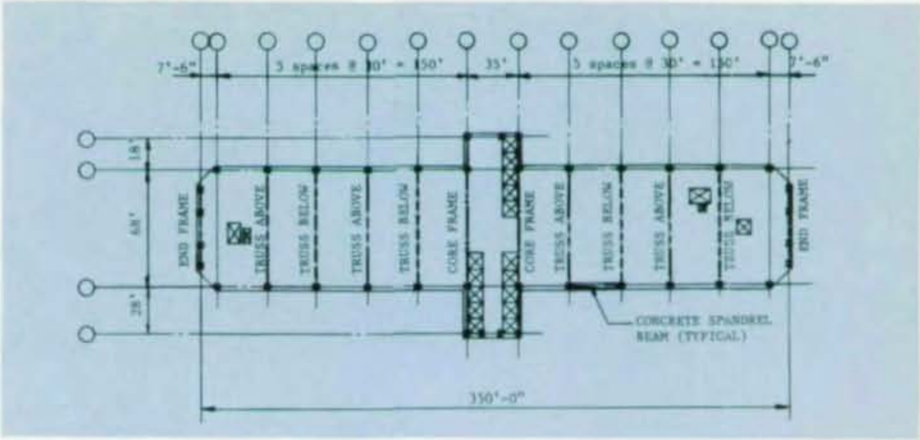


Fig. 2. Typical floor plan

Perspective of main entrance to Taj Mahal casino-hotel

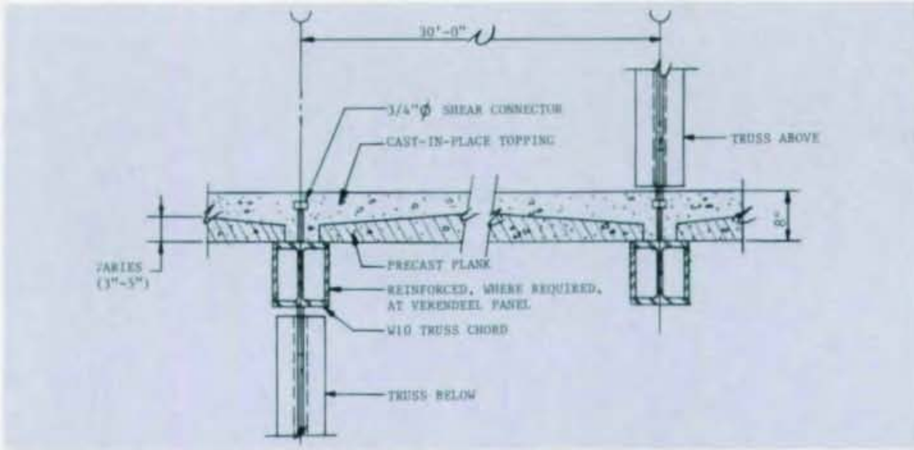


Fig. 3a. Typical bay section

through a series of tests. Both a pressure model, to measure local wind pressures, and a dynamic model to study the building's response were used then. A modified BOCA code wind was established in each direction which produced wind shears, moments and torsional effects equivalent to those indicated by the wind study. Average wind forces in the transverse direction exceeded 50 psf. Results of these tests helped to establish the design wind loads used in the final design.

One of the first and most critical design steps was the selection and design of the floor system. The most common and efficient system used in previous staggered-truss buildings is a hollow-core precast concrete plank with a thin cast-in-place concrete topping. However, in this instance, it became readily apparent that, a combination of building height and high wind forces would result in slab shears through much of the lower part of the building that would exceed the capacity of such a hollow concrete plank system. After study of several alternate schemes, a solid composite concrete-slab system was chosen. The system selected is an 8-in. thick precast, prestressed solid concrete slab with a cast-in-place topping. The precast units have a roughened top surface and extended horizontal ties which insure bond with the cast-in-place topping. The net result is a solid, composite slab which spans the typical 30-ft bay. At lower floors, with their large public live loads and high wind shears, a 12-in. total slab thickness was used.

The profile of the typical precast panel varies from about 3-in. thick at the ends to 5-in. thick in the middle (Fig. 3a). With the resultant section properties, the panel is capable of spanning the 30-ft bay unshored until, just prior to the casting of the topping, a minimal amount of shoring was applied to carry the wet concrete weight. This method greatly aided the erection process because trusses could be erected and the plank simply laid in place and erection-bolted to trusses. This requirement for minimal shoring meant several floors could be erected without the need for completed floors below to serve as a shoring base.

Tapered plank ends at the truss chords provide a further structural benefit. Since the horizontal shears carried down the building through the trusses are transferred through the slab, the point of transfer between the two elements is critical. In this case, ¾-in. dia. shear connectors welded to the top of the truss chord and embedded in the slab are used for this transfer. The tapered plank on both sides of the shear studs creates a large area of cast-in-place concrete at this point. The detail ensures proper embedment aiding in the shear transfer process. Architectural and other disciplinary restraints prevented the typical staggered-truss bent from being used on all column lines. Accordingly, three types of framing were employed: the typical staggered truss, a core frame and an end frame.

Staggered-truss framing system was used on typical transverse column lines. The lowest of the trusses is at the first or second level and the remaining trusses on every alternate floor. The top guest room floor, the 42nd, was to have provisions for large, open luxury suites, so all trusses were omitted at that level. The cumulative wind shear is so low at this high level that only the core and end frames were required to resist the wind. At the roof level, trusses of varying heights cantilever over the main building line to provide for the distinctive roof crown of the architectural design.

How Wind Shear is Handled

At the base of the building, the cumulative total of wind shear is brought into the foundation through the bottom chord of the lowest truss, which is embedded in a large concrete grade beam. The beam and truss connects the pile caps on opposite sides of the building. Diagonal braces were added at those column lines where no truss exists at the lowest level to distribute the lateral load evenly between all the building bays at the base. The diagonal braces transfer the lateral load of each alternate bay down to a steel beam embedded in the foundation similar to the adjacent truss bottom chord. Foundation piles are 165-ton capacity, 14-in. dia., steel shell, mandrel-driven, cast-in-place, concrete-filled. A 25-ft × 25-ft-8in. deep pile cap at the typical staggered truss bays contains 36 piles. The lateral capacity of the piles and cap system is sufficient to resist the applied wind shears.

Architectual features, corner balconies and fenestration prevented use of typical staggered truss in the two end bays of the building. However, torsional motion of such a long, thin buiding, called for stiff, lateral-resisting elements at the ends of the structure. To provide this stiff end, a 3-bay braced frame was introduced. The center bay of this 3-bay frame is diagonally braced with steel channels and then rigidly connected to deep, stiff, spandrel beams on each side connected to large columns. These end frames are short (45 ft) compared to the 68-ft span of the staggered truss. And once made stiff enough to attract their required share of the load, the exterior columns experience a substantial amount of uplift. To resist this uplift, the columns are embedded into the foundation, which consists of one large pile cap. This 100-ft long cap spans the width of the building and encases 110 piles. The large resulting moment arm of this long pile cap reduces the net uplift on the end piles to a minimal amount.

The center bay of the building, which contains the elevator core, presented another series of problems. Ten passenger and five service elevators would be needed to properly service this luxury hotel. The associated elevator shaft openings, in addition to other slab openings required for mechanical ducts, reduced to a fraction of its capacity the slab diaphragm so critical to the staggered-truss system. On one side of the core, the slab necked down to a thin 14-ft width through which all of the lateral shear would have to be transferred. At the lower floors, this proved extremely difficult. Several attempts were made to reinforce substantially this area, but this proved not feasible.

Modified Staggered Truss

A modification of the typical staggered-truss bent was required to solve the problem of the concentration of shear in this area. The result was a so-called core frame. It consists of a full floor-height truss spanning the building width. In the core, however, a truss is located at each floor level. Trusses at each level produce a lateral system of similar stiffness to that of the staggered truss. However, the presence of a lateral resisting element at each level means that shears which enter the system do not have to be transferred out. Therefore, the shears which must be transferred through the core slab are not the cumulative total of the shear over the building height but simply the shear of one story. It was found that the relatively small area of core slab remaining can easily carry this shear. Several of the trusses in the core at intermediate levels and at the roof were extended out to the elevator wings. By using these as belt-and-hat trusses, the building columns furthest apart are mobilized to give additional stiffness to these core frames.

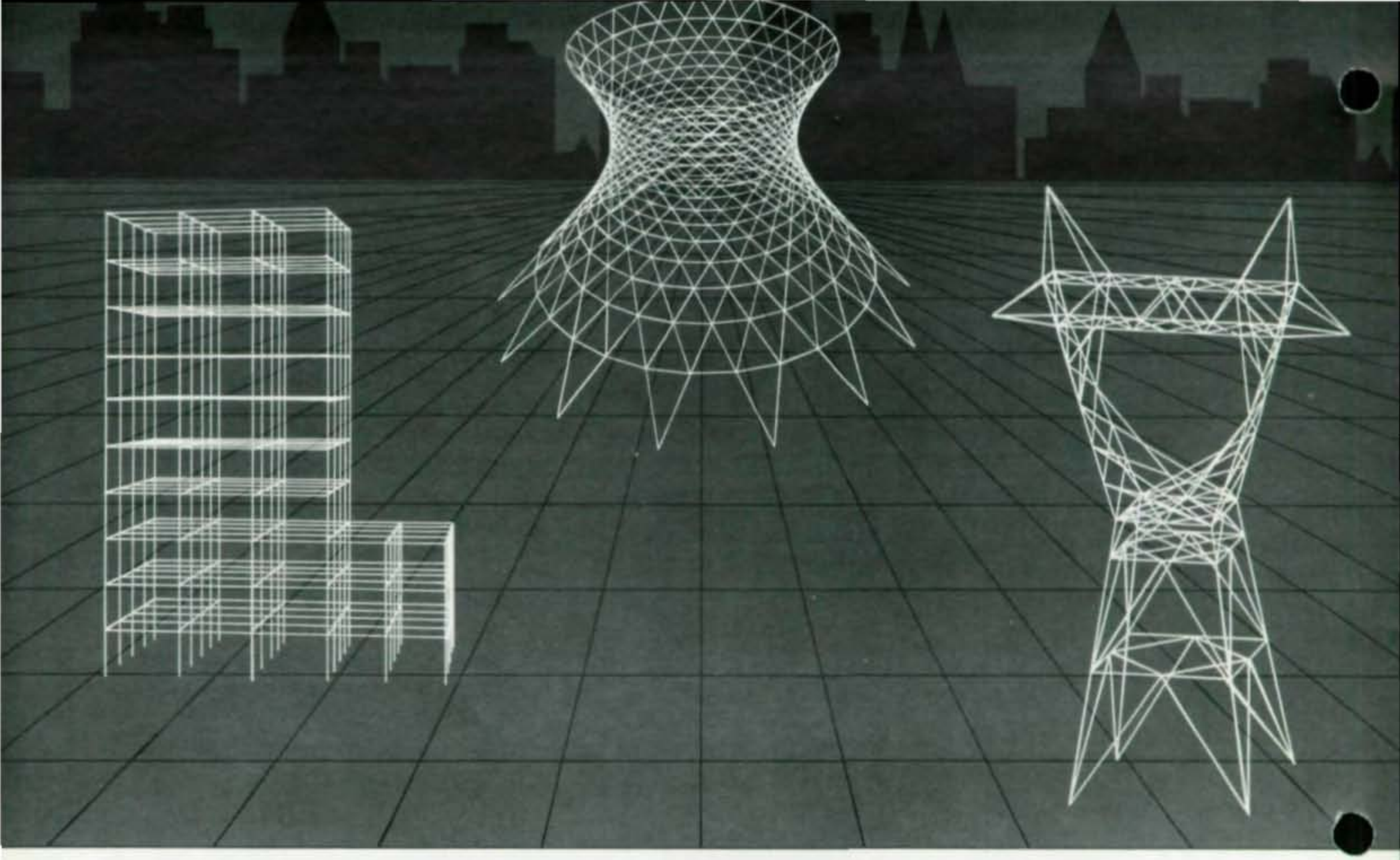
Once the exact configurations of the transverse frames were determined, computer models of them were generated. The various systems were linked together, and design loads applied. From this analysis of the relative stiffness of various elements, a more accurate determination of the shear flow through the slabs and into the trusses was attained. Combining this analysis with the relatively simple gravity analysis, the final design of the trusses was completed. In the typical staggered-truss bent, trusses at the lower public floors and at the roof were each specially designed. To help achieve economy of fabrication, detail and erection, trusses through the middle 30 floors were designed and detailed into only three different types, each covering 10 floors. The core and end frames were designed in a similar manner so as to achieve a maximum degree of economy through repetition. The core frame, with its solid wall of trusses, was to produce an erection sequence similar to the typical staggered truss. This was accomplished by having trusses on alternate levels shop-assembled, as were the typical staggered trusses. After these were erected, web members of intermediate levels were infilled in the field.

Computer Analysis for Final Design

Results of the computer analysis of the traverse lateral loading was used for the final design of the floor system. Employing a similar theory to that of the truss designs, slabs were designed in groups of 12-in. thick slabs of the lower floors and three separate designs of 10 floors each for the 8-in. thick slabs of the typical hotel floors. Gravity loads were added to the critical wind loads at each design level. The resulting stresses were checked and slabs reinforced as required. The typical slab bay throughout the hotel includes several duct and pipe openings at the center of the 30-ft span where back-to-back bathrooms meet. In addition, there are openings at the corners adjacent to the spandrels and the exterior columns. It has been proven that slab stresses are highest at openings and at the corners. An in-depth study of the typical bay was warranted. A detailed finite-element computer model and stress analysis was made. The results of the analysis were used in the final detailing of the slab system. Reinforcing quantities and patterns were determined at typical locations and at all openings.

Another related item, critical to the performance of the slab system, which was detailed during this analysis, was the plank-to-plank joint. This joint must maintain the integrity of the floor diaphragm and transfer in-plane forces from plank to plank. To minimize the amount of additional field work required, consideration of a welded joint (often used) was eliminated. Instead, a detail was devised whereby edges of adjacent plank taper down to a minimum 2-½-in.

Trusses cantilever up to 26 ft at roof level to form distinctive architectural crown.



thickness over a 6-in. width. The final result is a 1-ft wide section of cast-in-place concrete which is reinforced, to resist the forces through the joint (Fig 3b).

The spandrel beam is another integral part of the floor system, especially with regard to its diaphragm design as a deep beam. By connecting integrally slab to spandrel beam, the spandrel acts as the flange of the deep beam. This increases the lateral stiffness of the system, and has been shown to reduce stresses in the slab areas where they are known to be highest. However, since the spandrel beam is also part of the moment frame in the longitudinal direction, its design was based on the critical case of lateral loads in both directions.

Architectural as well as structural considerations were involved in the initial selection of the type of spandrel beam to be used. Both structural steel and concrete spandrel beams were considered. Both were capable of meeting structural requirements. Architecturally, an upturned concrete spandrel beam provided a fireproofed, smooth, finished surface to which interior window and sill details would adapt readily. The selection was a 12-in. wide by 4-ft deep stiff element that contributed to the rigid longitudinal frame. It also satisfied the architectural requirement. The combination of this stiff spandrel beam moment connected to the large building exterior columns and the relatively shallow floor-to-floor height created a rigid frame easily capable of relatively small longitudinal wind loads providing stiffness in that direction. And, at the lower levels, where an upturned spandrel beam could not be used, steel spandrels below the slab were combined with several bays of diagonal bracing. The system completed the lateral resisting system in the longitudinal direction which carries the wind shear down to the foundation.

Detailing of the spandrel beam went through several different stages. Among the options considered were: a steel beam encased in concrete, a cast-in-place concrete beam with mild reinforcing steel plus several versions of precast concrete beams, both prestressed and non-prestressed. The detail of its connection to the column was of far greater concern than the design of the beam. Tight erection tolerances inherent in the staggered-truss systems dictate beam-to-column connections that adhere to tight steel tolerances. Normal concrete construction tolerances were not acceptable during the erection of this system. The detail and fabrication of this beam had to adhere to the tolerances of structural steel. The final result (Fig. 4), was the combination of basic engineering design and advice from the steel erector. Similar, but smaller, versions of a precast concrete beam with a modified end connection have been used in previous staggered-truss construction. The concrete beam is shop-fabricated and reinforced with large (#14 and #18) rebars. The rebars are cadwelded to structural tees embedded at each end which are fitted with ¾-in. dia. shear studs. The rebar cutting and cadwelding was done by the structural steel fabricator, who also provided jigs to the precaster for setting cages. The final product was within steel tolerances and the field bolting of the connection to a shear plate welded to the column was performed smoothly on the job. Also, the floor slab was sufficiently locked into the spandrel by the topping pour through dowels and a shear key cast into the member.

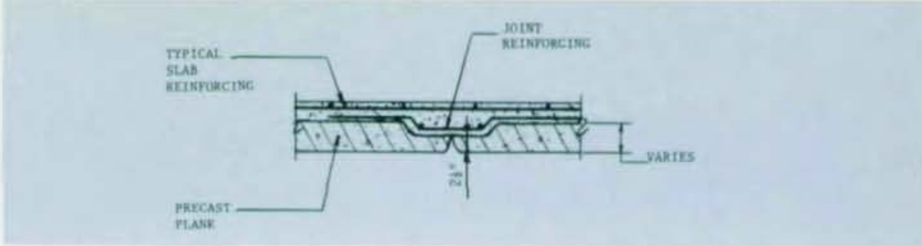


Fig. 3b. Plank joint detail

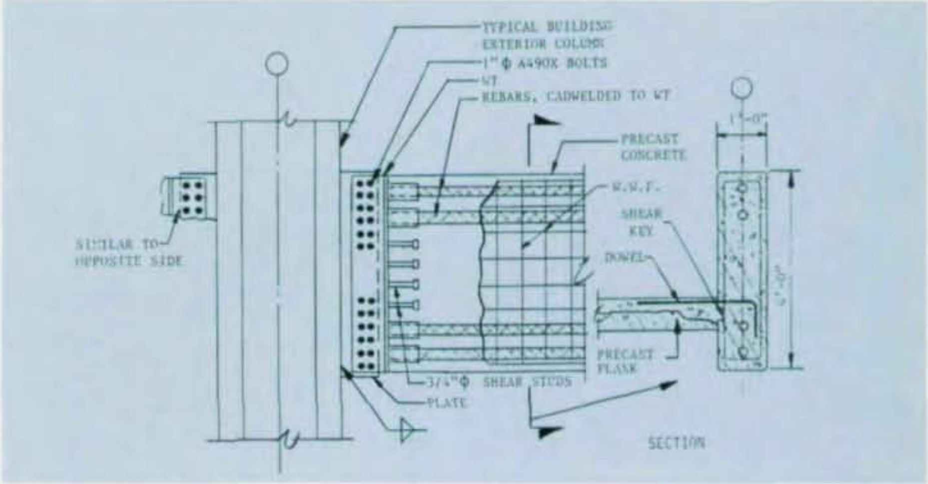


Fig. 4. Spandrel beam detail

After the design and detailing of major structural elements were completed, attention turned to the number of miscellaneous conditions throughout the building and coordination of those structural items with the other disciplines. Miscellaneous stair, duct and pipe floor openings were provided either by sufficiently reinforcing the slab or by framing openings with structural steel. In either case, it was imperative the diaphragm capacity of the slab system was not impaired.

The major mechanical systems run vertically through the structure and are accommodated by the slab penetrations. However, there are fire suppression and electrical elements which are required to run horizontally through each level. This presented some problems because of the relatively small floor-to-floor height and the inability of these elements to bend around the truss chords. The sprinkler pipe detail was solved by predetermining the pipe locations and shop cutting 4-in. dia. holes through the webs of the W10 truss chord members. Where required, the holes were reinforced with pipe or plates, thus insuring the integrity of the truss member. The main nonflexible electrical cable, which typically runs along tight to the underside of a slab, could not pass through the trusses. This situation was rectified with a relatively simple detail. Thin slots were cast into the bottom of the ends of the precast panels at predetermined locations. On top of the trusses two of these panels were set opposite from each other, and a small continuous slot was formed to the slab allowing passage of the cable over the chords. Other detailing problems were solved in a similar fashion.

Low-rise Convention Facility

The 44-story staggered-truss hotel is just part of a sprawling convention facility complex which surrounds it. The entire complex sits on almost 20 acres of land on two city blocks in Atlantic City adjacent to the Boardwalk. The complex includes more than 2,500,000 sq. ft of floor area. Estimated project cost is in excess of $500 million.

The low-rise convention facility surrounding the hotel has four levels and is framed with structural steel (mostly ASTM A572, Gr. 50) with a concrete-on-steel-deck floor system. Because of the owner's desire for a minimal number of columns in the casino (claimed to be the world's largest at 120,000 sq. ft), many long, clear spans were required. Typical bay size is 60 ft × 90 ft. In most of the building, the structural system employs a series of trusses spanning both the 60-ft and the 90-ft dimensions. Upper levels include ballrooms, meeting rooms and various other spaces of similar occupancy requiring high live loads which need sensitive truss designs. One-hundred twenty foot span trusses frame the roof over the main ballroom. Over 200 trusses are used throughout the low-rise structure, with depths ranging from 8 ft to 15 ft and weights from 7 tons to 56 tons.

Also included as part of the convention facility is a 63,000-sq. ft Exhibition Hall. This 300-ft × 210-ft structure has no interior columns and a 40-ft clear interior height. The roof is supported by large trusses spanning the entire 210-ft width, which vary in height from 21 ft at midspan to 16 ft at the ends and weigh in excess of 75 tons. The structure is scheduled for completion in the Summer of 1987. Total steel requirement for the project exceeds 28,000 tons.

Conclusions

Each individual project has its own set of particular parameters which generally dictate the selection of certain structure types and the elimination of others. Given the proper circumstances, as in the case reported here, the staggered-steel truss system has proven to be a desirable, economical structural system. The recent design and construction of the first high-rise buildings of this type have shown the system to be compatible with these, and even taller structures. The structure in this report is capable of efficiently resisting high wind loads and inherently exhibits the stiffness and dynamic levels required to maintain the strictest of occupant comfort levels. This fact was confirmed by the results of the wind tunnel dynamic test. They indicated that the structure was sluggish with low acceleration levels. While occasional modifications to the typical staggered-truss framing may be necessary to meet other requirements, this does not necessarily change the basic system behavior or efficiency. Details can be created that allow all the various systems of the total building to be integrated within the framework of the truss system. The net result is a highly efficient structural system which is relatively quickly and economically erected—and yields a high degree of architectural and planning flexibility. □

Architect
Francis Xavier Dumont
Atlantic City, New Jersey

Structural Engineer
Paulus, Sokolowski and Sartor
Warren, New Jersey

Steel Fabricators (joint venture)
Steel Structures Corporation (hotel tower)
Upper Saddle River, New Jersey and
Montague Betts Company (convention area)
Lynchburg, Virginia

Owner/General Contractor
Resorts International
Atlantic City, New Jersey

Michael P. Cohen ia a senior structural engineer and project manager with the consulting firm of Paulus, Sokolowski and Sartor, Warren, New Jersey.

SCADA/AMERICAD

Tailored to your best expectations

AMERICAD is a dynamic system specially configured to run the SCADA finite element analysis program efficiently.

Dynamic, because we build it for you when you order it. Not before. So your equipment has the most up-to-date specifications. And you have the competitive edge in a world of swiftly changing computer technology.

We've selected the highest-performance equipment consistent with maintaining strict quality control while satisfying the engineer's other computerized needs. We've also interacted with top engineering companies worldwide, gaining the invaluable insight that helps us create more-valuable systems.

Since 1978, American Computers & Engineers has completed installations for more than a thousand satisfied customers. Now we're looking forward to installing a SCADA/AMERICAD system for you. A system that's designed to meet your most critical demands. And built to live up to your most favorable expectations.

Call or write to learn more:

AMERICAN COMPUTERS & ENGINEERS

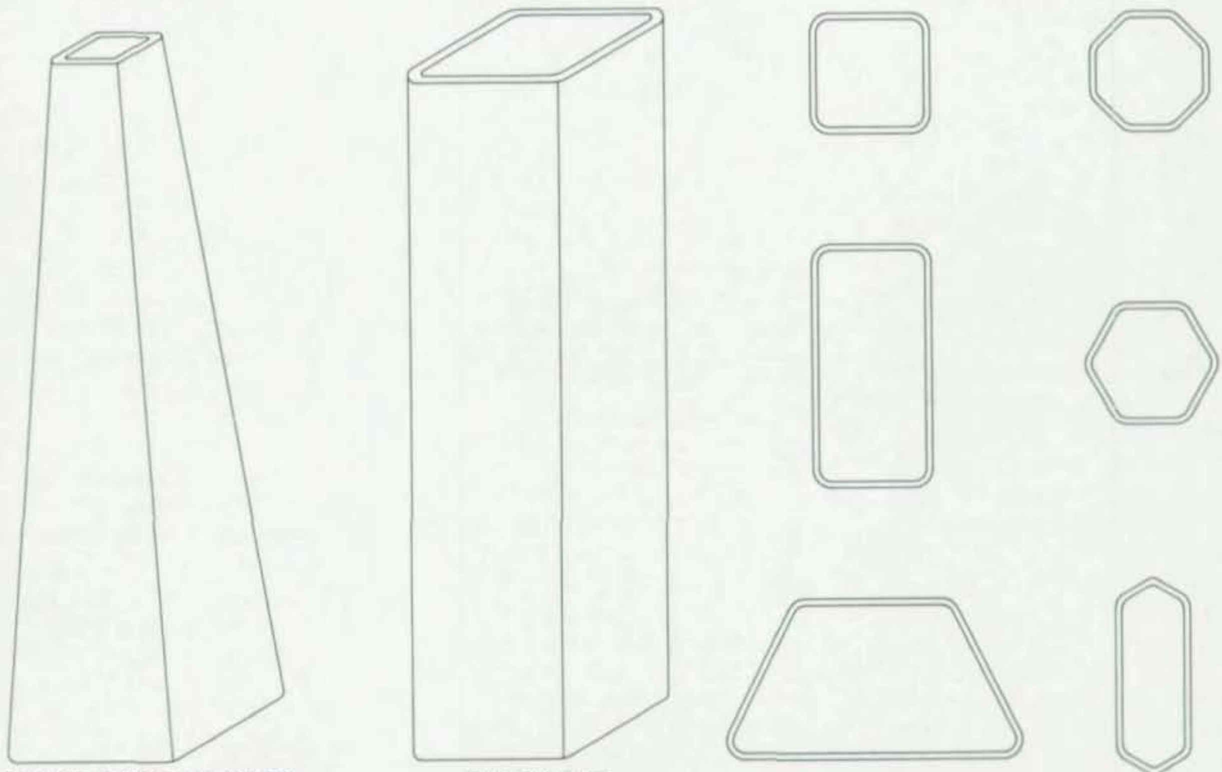
11726 San Vicente Blvd. • Los Angeles, CA 90049 • Tel: (213) 820-8998
CALCOMP • Anaheim, CA • Tel: (714) 821-2000
SIGMA DESIGN • Englewood, CO • Tel: (303) 790-9080

International:

AC&E, HELLAS • Athens, Greece • Tel: 671-9722
AC&E, LTD. • Glasgow, United Kingdom • Tel: 946-0181
KOZO, INC. • Tokyo, Japan • Tel: (03) 348-0641
IR. A. KOOIJMAN B.V. • Driebergen, Netherlands • Tel: 03438-15888
WILF ENGINEERING • Safat, Kuwait • Tel: 2-551-346

CUSTOM TUBULAR SECTIONS

MANUFACTURED TO MEET YOUR NEEDS



TUBULAR SECTIONS TO YOUR SPECIFIC SHAPE AND SIZE

- Sizes up to 26" square or 36" x 26" rectangular
- Tubes meet ASTM-A500 dimensional criteria
- Tapered or straight sections
- Sectional properties available on request
- Special radius corners when desired
- No minimum order size

GATE CITY STEEL®
THE GOOD SERVICE PEOPLE

GATE CITY STEEL - P.O. BOX 2620 - TULSA, OKLAHOMA 74101, ATTN. RON PAYNE
CALL TOLL FREE: 1-800-331-3002

Please send additional information about fabricated tubular sections.

NAME________ TITLE________ FIRM________

ADDRESS________ CITY________ STATE________ ZIP________

PHONE________

Steel Notes

LRFD MANUAL NOW AVAILABLE FROM AISC

Load and Resistance Factor Design (LRFD) Manual of Steel Construction, 1st Edition is now available. The 1,109-pg. Manual is based on the *Load and Resistance Factor Design Specification for Structural Steel Buildings,* effective September 1, 1986. The new design procedure relies on the actual strength of a member or component, rather than on an arbitrary calculated stress. Both working loads and resistance are multiplied by factors, and design is performed by comparing the results. The concept is intended to help engineers design steel-framed buildings of more uniform reliability, with more efficient use of structural steel.

The LRFD Manual is in six major parts: (1) Dimensions and Properties, (2) Columns, (3) Beams and Girders, (4) Composite Members, (5) Connections and (6) Specification, Codes and Commentary. The cost of the Manual (M015) is $42 for members and $56 for non-members, the Specification (S328) is available for $7.50 for members; $10 for non-members. Orders may be sent to AISC Publications Department, P.O. Box 4588, Chicago, Ill. 60680-4588. Visa & MasterCard are now acceptable. Please list your card number and expiration date.

A five-program lecture series scheduled for 63 cities will begin in December and extend through 1988. The seminars ae intended to introduce the LRFD Specification and Manual to design professionals. For further information about the seminars, contact Janet Manning, AISC headquarters, 312/670-5431.

NEW BOLT SPECIFICATION RELEASED

AISC has issued the new *Specification for Structural Joints Using ASTM A325 or A490 Bolts* (S329). The Specification, approved by the Research Council on Structural Connections of the Engineering Foundation, includes new provisions for "snug-tight," high-strength bolted connections. It defines material and shipping requirements, effects of overspray, proper use of installation methods and effective inspection. Five methods of installation are presented in the 48-pg. Specification. Endorsed by AISC and the Industrial Fasteners Institute, the Specification is $2.25 to members; $3 for non-members.

AISC MARKETING, INC. AIMS FOR IMPROVED MARKET

AISC Marketing, Inc. was formed to improve the market for structural steel by 600,000 tons annually. Staff engineers, strategically located throughout the U.S., provide steel structural design and cost analysis for bridges and non-residential buildings. Showing the economical design and application of steel for a particular project, the information is given to owners, developers, engineers, architects, contractors and construction managers engaged in selecting structural and framing materials. The company's goal is to analyze 200 projects in detail in the early planning stage and to track an additional 2,500 jobs.

The company, a joint effort by AISC, U.S.X. Corporation and Bethlehem Steel Corporation, has a current staff of 17 sales engineers. The Chicago-based organization is headed by Neil W. Zundel, president, and also AISC president. Ronald L. Flucker is vice president and general manager.

NEC AND COP TO MERGE IN NEW ORLEANS

The National Engineering Conference (NEC) and the Conference of Operating Personnel (COP) will be conducted simultaneously April 29-May 2, 1987 at the Rivergate Convention Center, New Orleans, La. Engineers attending will find the latest information on steel design, code changes, technological advances and recent research. Fabricators and erectors of structural steel can look forward to sessions on project management, shop and field inspection, safety, quality certification, welding, bolting, cleaning and painting. Topics of interest to both types of professionals will be: foreign competition, documenting change orders, responsibility for connection design, performance of structures and codes and standards. For one registration fee, attendees may participate in any of the joint conference sessions.

In addition to panel discussion/workshops, the conference will offer exhibits by firms who provide products and services to the steel construction industry. Over 1,500 attendees and exhibitors are expected for the "all-steel" conference. For information on exhibit space or the technical sessions, write to AISC, 1987 NEC/COP, P.O. Box 804556, Chicago, Ill. 60680-4107, or call 312/670-5432.

GUIDE PROVIDES INTRODUCTION TO LRFD

AISC's new *Guide to Load and Resistance Factor Design of Structural Steel Buildings* supplies background information needed for a successful transition from Allowable Stress Design (ASD) to Load and Resistance Factor Design (LRFD). The 68-pg. guide introduces the philosophy of LRFD, discusses major topics of the LRFD Specification and provides simplified versions of several equations for design of simple structures or components. The booklet is intended for engineers, architects and other design professionals who are not yet familiar with LRFD, or who need clarification of specific provisions. The guide is available for $10 from AISC's Publications Department, P.O. Box 4588, Chicago, IL 60680-4588. □

Steel Solution

WORLD FINANCIAL CENTER

Long-span Pedestrian Bridges Solve Access Problems

by Abraham Gutman, I. Paul Lew, Charles Thornton and Richard Tomasetti

The World Financial Center, containing about 7,000,000 sq. ft of office space, is being erected on the West side of lower Manhattan on a landfill between the bulkhead and the pierhead line of the Hudson River. The giant project literally is separated from Manhattan by the very wide Marginal Street. To unite the World Financial Center complex with Manhattan and its public transportation system, two long-span bridge structures have been designed.

The North Bridge, with a center span of 200 ft and an overall length of 352 ft, connects the second floor of the Winter Garden at the World Financial Center with the plaza of the Custom House at the World Trade Center (Fig. 1). The South bridge, with a center span of 220 ft and an overall length of 308 ft, connects the second floor of Building A at the World Financial Center with the south sidewalk of Liberty Street.

The bridge structures, designed to blend architecturally with the existing buildings, required large window openings. This was met by designing a structural system of two Vierendeel trusses with vertical members spaced 20 ft o.c. However, site restrictions due to the proposed construction of the new West Side Highway limited the points of support for the proposed bridges. This factor called for design and construction of two of the largest Vierendeel pedestrian bridge structures in the world.

Fig. 1. North Bridge at World Financial Center

Design Concept

The challenge was to design two pedestrian bridges with minimum pier supports and temporary supports so as not to interfere with street traffic during construction. To achieve this, one pier had to be placed in the World Trade Center area where it would penetrate five existing stories of highly stressed floors to a new foundation. To limit construction at the site, and street closings, shop fabrication of large structural units was implemented. After several structural concepts, such as two Warren trusses and platform plate girders, were considered, a Vierendeel truss system was selected. This system fully responded to the architectural requirement of large and open windows. And it was much more economical than a platform plate-girder system.

Each bridge was designed with a clear height of 17 ft to make pedestrians feel comfortable along the nearly 400-ft walk between the two buildings. To emphasize the openness of space, verticals in the Vierendeel truss were placed 20 ft o.c. to permit design of 15-ft × 15-ft large windows running along the length of the pedestrian bridges (Fig. 2). These design parameters resulted in a Vierendeel truss system with fewer joints but with higher moments and shears.

A Vierendeel truss has its minimum moments and shears at midpoints between the rigid joists. This unique characteristic of this truss resulted in placement of field splices at midpoints be-

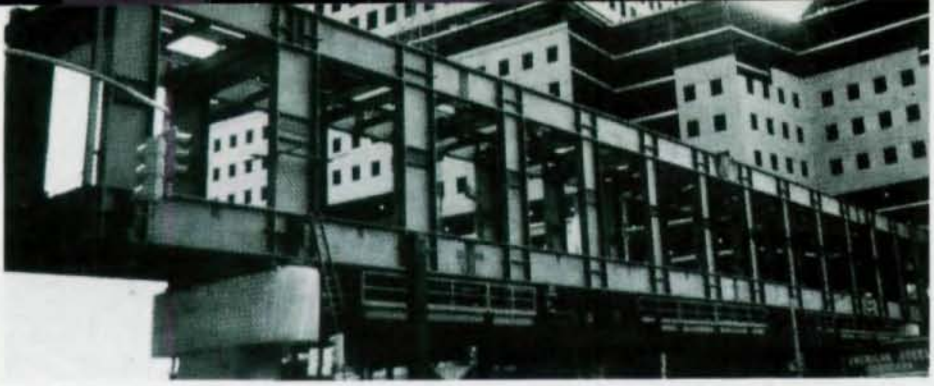


Fig. 2. Verticals in truss permit large windows

tween joints and therefore, large truss sections—14 ft-6 in. high, and 80-ft long—were designed to be shipped to the construction site. This concept minimized the number of temporary towers, eased field erection, and, most importantly, permitted critical fabrication in the shop.

Design

The truss members consisted of wide-flange plate girders, with flanges 24-in. wide and varying in thickness from 1 to 3 in. Webs were 60-in. deep and 1-in. thick. The critical aspect of a Vierendeel is the design of flange-to-web welds within a member and flange-to-flange welds between truss members. These welds are important because of the very short distance in which the welds can be developed. In the case of the North and South Bridge, with its 60-in. plate girders and over 200-ft main span, the development length averaged 90 in. in the chord members.

There are two critical areas in the weld design of Vierendeel trusses; one is the joint itself and the other is the area between the joints at the midpoint of the truss members (Fig. 3). At the joint, the moments have to be balanced. The verticals have to balance the moments in the two adjoining horizontals. As a result, all forces in the flanges of the members have to be developed at the joint. The joint welding requirements were very carefully detailed on the drawings.

Independent of the long-span Vierendeel action, transverse stability of the bridge had to be achieved. The bridge, over 30-ft high, is subject to high transverse winds. Each vertical member of the truss was laterally braced by creating moment frame rings with transverse floor beams that span between trusses. The floor of each bridge were X-braced horizontally to provide overall lateral stability.

Fabrication

The Vierendeel trusses were designed with the fabrication process in mind. First of all, by specifying all shop-welded joints, welding was performed under optimum conditions. Second, to maintain ductility of A36 steel, fine-grained, silicon-killed steel was specified. Thirdly, to assure that welding quality was maintained, the AASHTO provisions for Fracture Critical Steel were specified with notch toughness requirements for ductility.

The most important weld occurred at the T-joint connecting the flanges of the verticals and horizontals. This joint was accomplished by providing a full shop-penetration weld. To assure that welding access holes for full-penetration welds would not produce stress risers, the design drawings indicated full-scale access of the hole details. The fabricator built a full-scale mockup of the joint to verify procedures used during the fabrication. The backup stiffeners were welded with fillet welds to limit restraint on them. All

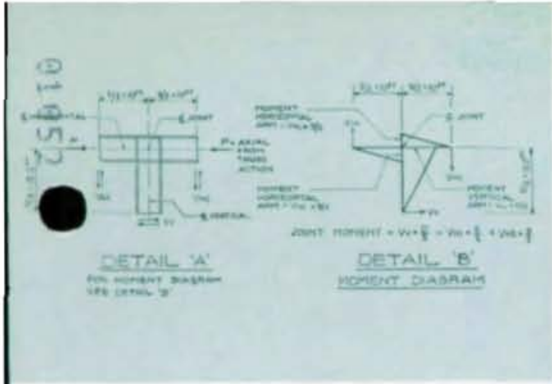


Fig. 3. Typical Vierendeel truss section

welds were done by using submerged arc welding with a low hydrogren electrode to assure high quality control.

Terminus Design

The design of the East Terminus of the South Bridge presented a unique challenge. It housed an escalator which serves as a connecting link between the South Bridge and Liberty Street sidewalk. Because of site restrictions, the East Terminus was only 9 ft-8 in. wide. This width limitation, in turn, imposed severe restrictions on the supporting structure.

The two main plate girders on either side of the escalator were 4 ft-6 in. deep, with flanges only 7.5-in. wide. The plate girders spanned 85 ft and followed, in general, the profile of the escalator, i.e., it had a 40-ft sloped section and a 45-ft horizontal section. The main design problem was the bracing of the top flange of the girders. To solve this problem, a welded pony-girder system was employed. This system consisted of a U-shaped welded frame running perpendicular to the main girders. The verticals of the U-shaped bracing frames are double pairs of welded stiffeners with a closed end that creates a box cross section. The box vertical section was then welded to an 8-in. × 14-in. horizontal box beam to form a U-shaped bracing frame. Horizontal X-bracing provides overall stability.

Erection

Most of the steel erection was performed on weekends and employed a minimum number of temporary towers. Bottom sections 14 ft-6 in. high by 80-ft long were shipped to the site and placed on the temporary towers and permanent concrete pier supports. Top sections were delivered next to the site and field spliced at midpoints. This procedure was repeated until the Vierendeel trusses became self supporting and the temporary towers could then be removed. Temporary underbridge decking was installed quickly to permit street traffic to proceed as the infill floor pieces were installed.

The erection technique developed for the Vierendeel trusses exceeded expectation in the field. By shipping large shop-fabricated sections to the site, the field erection of the pedestrian bridges disrupted city traffic minimally by being accomplished in eight weekends. □

Architects
Cesar Pelli Associates
New Haven, Connecticut and
Haines Lundberg Waehler
New York, New York

Structural Engineer
Thornton-Tomasetti, Inc.
Office of Lev Zetlin Associates, Inc.
New York, New York

Steel Fabricator
Standard Structural Steel
Newington, Connecticut

Owner
Olympia & York
New York, New York

Abraham Gutman, Paul Lew, Charles Thornton and Richard Tomasetti are associates in the engineering firm of Lev Zetlin & Associates, New York, New York.

Fire Protection

DESIGN AIDS FOR FIRE PROTECTION

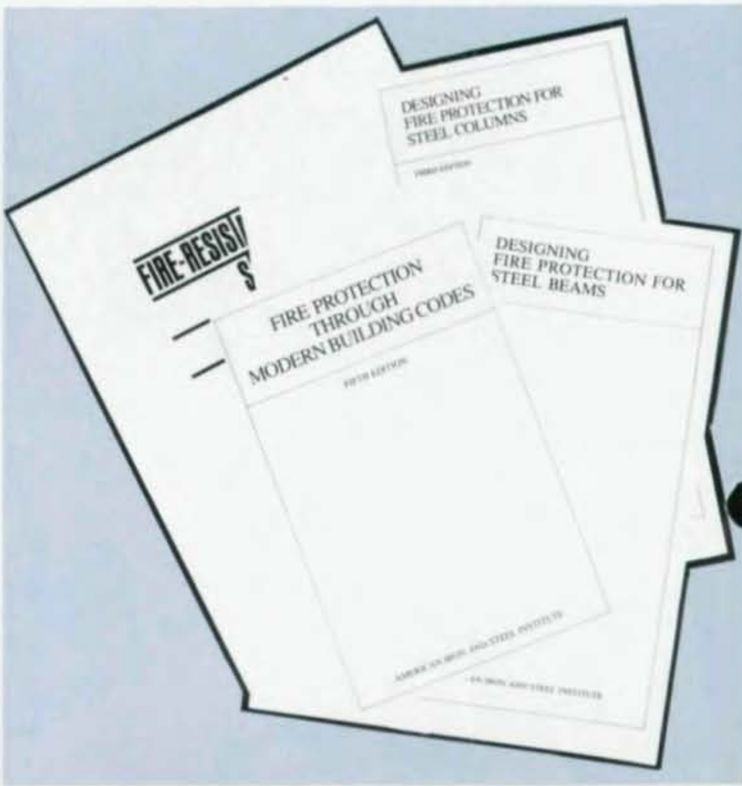
by Nestor Iwankiw

Fire in buildings is an unpredictable and often tragic occurrence which cannot be totally prevented. However, its catastrophic effects can be mitigated and controlled through good engineering and planning. In this sense, fire may be considered as another uncertain extreme event for which a structure must be designed, much like earthquake or tornado loads.

Since fire safety is a major public issue, U.S. Model Building Codes impose certain requirements to minimize both loss of life and property damage. Steel possesses the desirable characteristic for a building material of noncombustibility. Thus, it does not contribute additional fuel to the fire, such as timber, or produce toxic gases, such as plastic. However, its loss of strength and stiffness at elevated temperatures could pose a problem relative to structural integrity. ASTM E-119, the accepted American test standard for structural fire resistance, subjects a test assembly to a controlled time-temperature fire exposure in a furnace. The temperature and deflections of the assembly under maximum design load are monitored. A limiting member temperature and stable deflections are the main criteria that determine a resistance rating time. The building codes reference this empirical information to specify a minimum fire-resistance rating in terms of hours for a given building, based essentially on these factors:

1. Type of occupancy
2. Building size—height & area
3. Fuel potential
4. Building location

A rigorous analysis of fire effects on a building is a formidable task which requires not only accurate knowledge of the duration and intensity of the fire exposure, but also the heat transfer to the structural system—together with the structure's nonlinear response, including the effects of temperature induced strains and thermal degradation of material properties. This multiplicity of variables and disciplines had hindered the development of analytical models and constrained fire protection toan almost exclusively empirical science. Now, as a greater understanding of this fire-load environment and structural fire resistance has evolved in conjunction with modern computer technology, more complete and rational engineering design methods for fire protection are coming to fruition.



Standardized E-119 test results still serve as a useful benchmark of expected fire resistance, even though they do not correlate directly with actual building construction or fire conditions. Simplified computational procedures can then extend these limited experimental results to other structural configurations.

The increased awareness of the validity of logical engineering analyses has gained wider acceptance for analytically derived steel fire protection methods. This trend is expected to continue as public confidence grows in the reliability of structural fire analysis, and as the cost penalty and crudeness of the purely empirical approach is more fully realized.

The AISI has been most active in meeting the public fire safety need by sponsoring steel fire protection research, preparing suitable publications and recommending appropriate build-ing code changes. The objective of this presentation is to summarize the currently available literature on steel fire protection design, its contents and practical applications.

Fire Protection Through Modern Building Codes

The 1981 5th Edition of this paperback, 234 pages, is available through AISI. It includes extensive references to the major building construction and fire protection standards. The central issues covered are fire severity, fire hazards relating to occupancies and building size, structural fire protection and means of egress. A complete chapter is devoted to structural fire endurance, including the standard fire test for the main structural members (beams, columns, walls) and the common fire protection materials. This book can be of great assistance in understanding fire-related building code requirements and their rationale. It also provides good general introduction to the more detailed AISI booklets to be discussed subsequently and an overview of the current state of the art. The book is $5 per copy from AISI.

Fire-resistant Steel-frame Construction

An updated 1985 3rd edition handbook is being prepared by AISI. The purpose of this publication is to provide architects, structural and fire protection engineers and building officials with usable, current data for design evaluation and planning of fire-resistive steel structures. It covers the various topics of steel fire protection with additional information to complement the fire endurance chapter in the previous publication. Again, fire-protection materials and methods, fire-resistance ratings for beams, columns, floors, roofs, trusses and exposed steel members are discussed. An overview and outside references on repair of steel after a fire are presented. Also included is a section on liquid-filled columns.

Fire-resistant Steel Frame Construction may be obtained from AISI.

Fire-resistance Directory

This book is recognized by the building code officials as the main authoritative reference on fire-resistance ratings for building designs. It is updated annually by Underwriters Laboratories (UL), an independent testing laboratory, and lists hundreds of specific assemblies tested in accordance with ASTM-E119.

The Model Building Codes all accept the UL fire-rated assemblies that include floor-ceiling designs, roof-ceiling designs, beam designs, column designs, truss protections, and wall and partition designs. Each fire-rated design describes in detail only the individual assembly components (and their arrangement) that were either actually used in the fire test or approved through a later special study. Thus, the many other possible variations or substitutions must be rationally evaluated. The general design information section of this directory gives some guidance on how to treat the common deviations from the listed rated assemblies. For example, a given floor-ceiling design will list one steel beam size with a certain fire protection thickness requirement. Simple formulas developed by AISI are shown in this directory as an acceptable method for substituting other steel members with appropriate protection thicknesses.

The *Fire-resistance Directory* is sold by Underwriters Laboratories. The previously described AISI booklet—*Fire-resistant Steel Frame Construction*—contains an excellent summary of the different UL fire-rated constructions and is keyed to the UL design numbers in the directory. This index enables one to more easily locate in the extensive UL

Geometric Data Flow's Detail™ Feature List

Simply Input

- Uses grid planes from design instead of absolute X, Y, Z.
- Multiple member input instead of one at a time.
- Global assignment of connections, not point by point.
- Produces opposite hand structures with minimal effort.
- Interactive CAD graphics for drawing modifications.
- Easy to learn (8-10 hours training required).

Input Verification

- Plotted framing plans and elevations showing the position of members and the connection types assigned to each connection point, prior to producing shop drawings.

Drawings and Reports

- Automatically details all connection material separately.
- Automatically details all members (beams, columns, etc.) by size and weight, combining all identical members.
- Automatically plots bills of material, weights.
- Automatically generates field bolt lists.
- Automatically assigns and plots shipping marks for erection.
- Automatically generates advance bill of material.
- Automatically generates load lists for sequencing.
- Automatically generates a mill order for most economical lengths.
- Automatically plots full size templates.

Comprehensive Concept

- Details the building as a unit instead of member-by-member.
- Performs connection design calculations.
- Flags troublesome connections, allowing the user to make corrections prior to plotting detail drawings.
- Maintains a complete file of structural members and connection material for estimation applications.
- Integrates to virtually any shop CNC equipment.

FOR MORE INFORMATION:

GEOMETRIC DATA FLOW, INC.
337 N. Vineyard Avenue, Suite 206
Ontario, California 91764

OR CALL TOLL FREE:

1-800-DETAILS (338-2455) IN CALIFORNIA: 1-714-984-1269

Directory the exact fire-rated assembly number for a particular construction.

Designing Fire Protection for Steel Columns

The 1980 3rd Edition is also available (free) from AISI. A revision is currently being prepared. This publication describes the main factors that affect steel column fire resistance relative to the ASTM E-119 test standard: type of fire protection and its thickness, weight of steel column and its heated perimeter. Through the years, many UL tests have been conducted on steel column sections. From this large and representative data base, simple formulas were developed by AISI to predict the equivalent E-119 fire rating of most column sizes and protection thicknesses encountered in design practice. Separate prediction equations are given for gypsum wallboard, spray-applied, or concrete encased columns due to the distinct properties of these protection materials. For a constant rating, an inverse relationship exists between protection thickness and weight-to-heated perimeter ratios. The Model Building Codes have adopted the AISI equations for columns protected with gypsum wallboard and concrete and for W-shape columns with spray-applied cementitious and mineral fiber products.

To determine fire resistance of steel columns protected with spray-applied cementitious and mineral fiber materials, the following formula is used:

$$R = \left[C_1 \left(\frac{W}{D}\right) + C_2 \right] h$$

Where

R = fire resistance (minutes),
h = thickness of spray-applied protection (in.),
D = heated perimeter of the steel column (in.),
W = weight of steel column (lb./ft), and
C_1 & C_2 = material-dependent constants determined for specific protection materials by the ASTM E119 Standard Fire Tests.
= 69 & 31, respectively, for cementitious material
= 63 & 42, respectively, for mineral fiber material

A similar formula for columns protected with gypsum wallboard produces the graph shown in Fig. 1, as an illustration of the functional relationship of the main fire resistance parameters.

Weight-to-heated perimeter ratios for typical wide-flange column shapes (both contour and box protected) are listed in the publication for use in these formulas. Also, design tables are shown for protection thickness requirements for many column sections for various fire resistance ratings (see Table 1). The only limitation of this calculation method is that fire protection for any column sections heavier than a W14×233 (the largest tested to date) be kept the same as for the W14×233. An AISI sponsored UL fire test on a jumbo column is planned for 1986 to extend the range of applicability of the prediction equation.

Designing Fire Protection for Steel Beams

Similar to the fire protection problem with column members, seldom do steel beams and

Table 1. Calculated Thickness of Spray-applied Cementitious Material on Typical W-shape Columns

Structural Shape Designation	Fire Resistance Ratings			
	1 hour	2 hours	3 hours	4 hours
	Total Thickness (in.)			
W14×233	½	¾	1	1¼
×193	½	¾	1¼	1½
×159	½	1	1¼	1¾
×132	½	1	1½	2
×109	¾	1¼	1¾	2¼
× 90	¾	1¼	1¾	2½
× 68	¾	1¼	2	2½
× 61	¾	1¾	2	2¾
× 48	¾	1½	2¼	3
W12×190	½	¾	1	1¼
×136	½	1	1¼	1¾
×106	½	1	1½	2
× 87	¾	1¼	1¾	2¼
× 72	¾	1¼	2	2½
× 65	¾	1½	2	2¾
× 53	¾	1½	2¼	2¾
× 45	¾	1½	2¼	3
W10×112	½	1	1¼	1¾
× 77	¾	1¼	1¾	2¼
× 54	¾	1½	2	2¾
× 49	¾	1½	2	2¾
× 39	¾	1½	2¼	3
× 33	1	1¾	2½	3¼
W 8× 67	½	1	1½	2
× 40	¾	1½	2¼	2¾
× 31	1	1¾	2½	3¼
× 28	1	1¾	2½	3½
× 24	1	1¾	2¾	3½
× 18	1	2	3	3¾
W 6× 25	1	1¾	2½	3¼
× 20	1	1¾	2¾	3½
× 15	1	2	3	4
× 9	1¼	2¼	3½	4½
W 5× 19	1	1¾	2½	3¼
W 4× 13	1	1¾	2¾	3¾

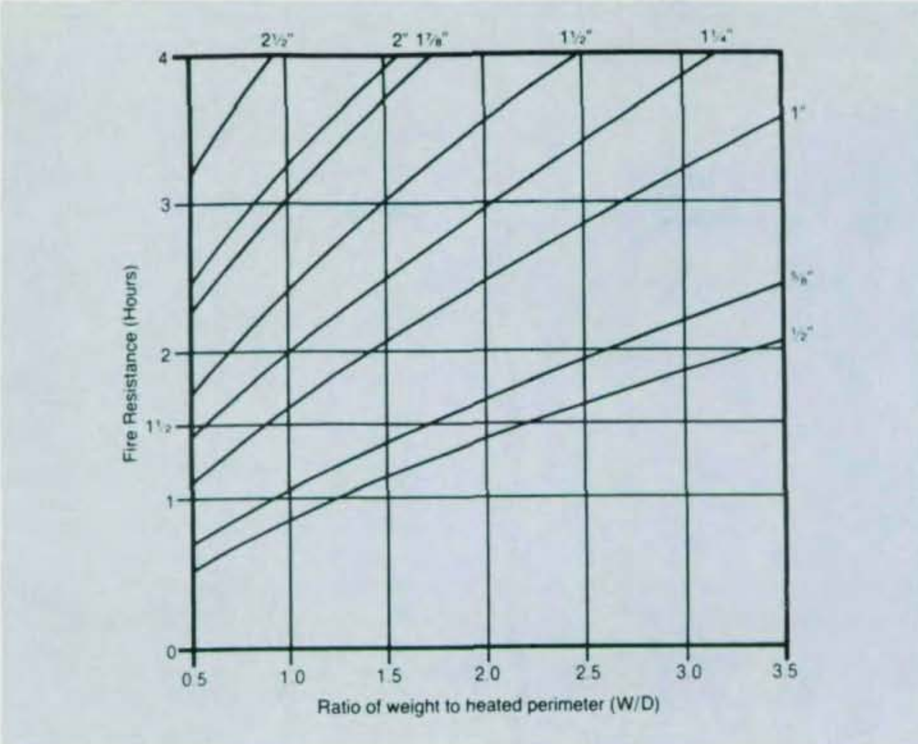


Fig. 1. Fire resistance of structural steel columns protected by various thicknesses of gypsum wallboard

girders in an actual building structure exactly match a listed UL Design in the *Fire-resistance Directory*. Thus, a member substitution equation for spray-applied protection was developed to accomplish this necessity. Again, an approximation formula was derived from the many experimental E-119 results. This formula is also given in the UL *Fire-resistance Directory*. It relates weight-to-heated perimeter ratio and spray-applied protection thickness of the substitute steel beam to the beam shown in the UL Directory design:

$$h_1 = \left[\frac{W_2/D_2 + 0.6}{W_1/D_1 + 0.6} \right] h_2$$

where

- h = thickness of spray-applied fire protection (in.).
- W = weight of steel beam (lb./ft).
- D = heated perimeter of the steel beam (in.).
- subscript 1 = refers to the substitute beam and required protection thickness.
- subscript 2 = refers to the beam and protection thickness specified in the individual UL Design.

Use of this equation is subject to these limitations:

1. The equation applies to beams having W/D values not less than 0.37,
2. h_1 cannot be less than ⅜ in., and
3. The Unrestrained Beam Rating in the UL Design is not less than 1 hour.

The AISI booklet—*Designing Fire Protection for Steel Beams*—provides more explanation of this beam substitution method, its application in several examples taken from the three Model Building Codes and a tabulation of the weight-to-heated perimeter ratio for the standard wide-flange beams. There is now no valid technical reason for a maximum uniform thickness of protection on all the steel members of a frame.

In addition, this booklet discusses the meaning of restrained and unrestrained ratings and the effect of applied stress level on fire resistance. The E-119 fire test methods correctly categorize "steel beams welded, riveted, or bolted to steel framing members (as) restrained." In addition, the AISI-developed computer program FASBUS II provides an engineering basis in support of E-119 that steel floor systems are restrained. Current AISI research is aimed at developing more specific engineering aids on this subject for engineers and building officials. The construction cost savings from justifiably eliminating excessive steel fire protection by these analytical methods can be significant. Again, this booklet is available from AISI.

Designing Fire Protection for Steel Trusses

This AISI publication extends the uses of existing ASTM E-119 test data for steel columns and beams to meet performance standards for trusses. This is necessary because furnace size limitations prevent direct fire testing of assemblies with large elements, such as trusses, under representative installed conditions. A conservative approach for establishing truss fire ratings has been accepted by most code authorities and is described in this booklet.

As with other structural elements, there are three basic means to achieve fire protection:

1. Ceiling membrane insulations system
2. Enclosing truss members separately, usually with some spray-on material
3. Enclosing the entire truss assembly in an envelope, usually gypsum wallboard

The type of truss fire-protection requirements is dictated in the building codes by its structural function. There are three categories of trusses: transfer, staggered and interstitial. The first two truss systems carry loads from two or more floors and require individual fire protection. Interstitial space trusses, on the other hand, carry only one floor load and, thus, may be protected by any of the three accepted methods, the simplest being the ceiling membrane. Hence, fire-resistant designs for interstitial trusses can be based conservatively on UL tested assemblies with open-web steel joists and suspended ceilings.

For the individual protection requirements, the steel column calculation formulas described previously can be conveniently used. The fire exposure conditions for the most critical truss elements are essentially the same as the test exposure for individual column elements. In the case of staggered trusses, gypsum wallboard enclosures are based on E-119 wall tests conducted with lightweight steel elements in the wall cavity. The temperature limitation on the lightweight elements is based on E-119 temperature criteria for steel columns.

The updated second edition is also available from AISI.

Fire-safe Strutural Steel—a Design Guide

While the previously mentioned standard test methods have long been in existence for interior structural members, there is no comparable experimental procedure for evaluating the fire endurance of exterior unprotected structural steel. The methods described in *Fire-safe Structural Steel* are analytical. They have been substantiated by much test data developed in the U.S. and in other countries where exposed steel structures have been built. The analysis is based on classical theoretical principles of physics and heat transfer which, when governed by the limiting steel temperatures, such as those in ASTM E-119, produces a reliable methodology consistent with that used in building codes for interior members.

The primary analytical considerations for exterior steel is the fire exposure based on burning area dimensions, fire load, rate of burning, flame geometry, flame temperature in proximity of the exterior steel member, the heat transfer to and heat loss from the exterior steel member, and finally, the steel temperature at the critical point. Convenient step-by-step worksheets are highlighted in this booklet, together with relevant numerical tables and graphs needed to perform the calculations. In addition, conservative design recommendations are offered for preliminary use, such as spatial separation and flame shielding.

This booklet is available from AISI.

Conclusions

AISI and the steel producers have provided much dynamic leadership and technical expertise towards the improvement of steel fire-protection designs. First, the results of numerous standardized E-119 tests at UL have been translated into convenient formulas that permit a wide substitution of steel members for those listed in UL fire-rated designs. AISI is now field testing a recently completed software package written in Microsoft BASIC language used by "IBM Compatible" computers. The equations used in the software program are exactly the same as those found in the AISI publication on columns, beams and trusses. In addition, research funded by AISI is progressing closer to the final goal of a general structural fire analysis model which will not only be capable of simulating a given E-119 test, but also of performing a fire analysis of an entire building under actual conditions.

These major accomplishments are making structural steel building frames more economically competitive while maintaining adequate safety levels. Therefore, AISC recommends use of these recent fire protection technology advancements. Only a balanced understanding of structural fire response, through analysis as well as tests, can lead to safe, functional and cost-effective buildings.

All the publications referenced in this article, with the exception of the UL *Fire-resistance Directory*, may be obtained directly from AISI, at nominal cost: American Iron and Steel Institute, 1000 16th Street, NW, Washington, DC 20036, (202) 452-7194.

Acknowledgement

The AISI publications referenced in this paper were prepared under the direction of the Subcommittee on Fire Technology/Committee on Construction Codes and Standards in cooperation with the Committee of Structural Steel Producers and the Committee of Steel Plate Producers. Extensive contributions to the substantive content of these design aids were made by Richard G. Gewain and AISI regional director, Delbert F. Boring. Their review and comments on this summary article are appreciated. □

Nestor Iwankiw is director, Research and Codes, AISC, Chicago, Illinois. Reprinted from the Proceedings of AISC's National Engineering Conference, June 1986.

Fast Tracking

PUROLATOR SUPERHUB

Under Budget, Ahead of Schedule

by Edward H. Mahoney, Jr.

Design and construction of Purolator Courier's new \$45-million "superhub" air distribution center in Indianapolis, Ind., can be characterized by a term synonymous with the firm's way of doing business. It was fast!

In about nine weeks from the start of design, all major bid packages for the primary structures (a 348,000-sq. ft sort building for air distribution activity and a 19,000-sq. ft line haul building for truck docking) were issued—including packages for fabrication of nearly 3,000 tons of steel. Ten weeks later, design of all support structures had been completed and construction had begun.

What was required to accomplish these tasks (as well as the completion of facility construction under budget and four months ahead of schedule) was a fast-track approach involving the close coordination of engineering and construction and the careful sequencing of bid packages. It also involved the complete cooperation of the owner in making and adhering to important design decisions that would allow fast tracking to take place. A difficult job, but it worked well.

Structural Concepts

For economy of construction and overall flexibility throughout the entire facility, both mild- and high-strength structural steel (ASTM A36 and A572) were chosen as the material for the superstructure. The A572 steel was used for some columns and all mezzanine steel. The use of steel resulted in rapid fabrication and erection and was a key to the successful completion of the project. Because of the elevated conveyors and extensive amount of mechanized sorting equipment, careful attention had to be paid to internal building loads on the mezzanine steel, roof steel and special support structures. For additonal strength and economy, mezzanine beams and girders supporting concrete slabs were designed compositely.

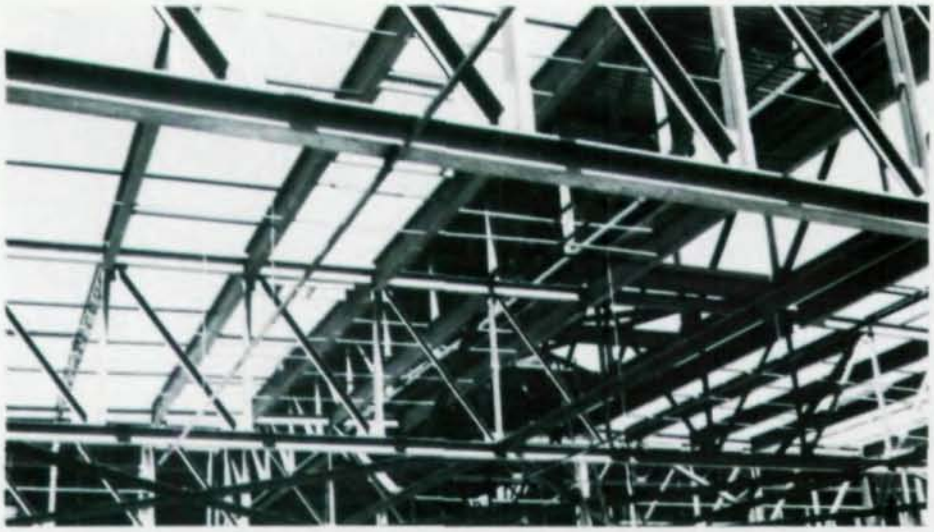
The sort building is designed so it can be lengthened from either end, mezzanines can be added in any bay, and material handling equipment expanded throughout the building. In the narrow direction of the building, rigid frames consisting of Pratt trusses and deep columns carry lateral loads to the foundations. In the long direction, horizontal trusses at the bottom chord level and wall bracing are used for lateral stability. Bay spacing is 49 ft-4 in. × 30 ft. Because of the nature of the facility, exterior walls were designed to resist blast pressures from jet engines.

The line haul building incorporates 70-ft long, 7-½ ft deep, clear-span Pratt trusses as primary roof framing members. Because of the number of truck doors (95 in all) lateral stability was accomplished by using rigid frames in both directions. The 115-ft long conveyor bridge, which connects the sort building and line haul building, was supported on the roof trusses.

Structural steel framing on line haul building

Four-building Facility

The Purolator facility, a 400,000-sq. ft, multiple-building complex serves as the firm's domestic hub for overnight small-package delivery of domestic and international shipments. The sort



Extensive use of structural framing looking toward sort building

building is 204 ft × 990 ft, with a roof height of 38 ft. Ground level and mezzanine space are for sort operations utilizing extensive automated materials handling. Additional mezzanine space houses administration, air and ground operations management, flight control and pilot support facilities. Flight control operations are in a tower extending above the roof of the building. The line haul building, connected to the sort building by a second-story conveyor bridge, includes 37, 13-ft wide truck dock positions as well as administrative and support facilities. The conveyor bridge will enable the routing of packages received by air to trucks, and vice versa.

A guard station/employee entrance adjoins the sort building and is connected by a pedestrian walkway bridge. A 13,000-sq. ft ground vehicle maintenance facility is on the perimeter of the site. In addition, a fuel farm includes two 10,000-barrel tanks for storage of 850,000 gallons of jet fuel and an adjacent structure for bulk storage of a chemical used to deice aircraft.

The advanced materials handling system, which will sort 150,000 packages per day, is designed for expansion to more than double that capacity.

Architectural Concepts

Building materials and techniques were derived from consideration of local environmental factors, energy and construction costs, maintenance characteristics and the fast-track schedule. In addition, while a relatively low-cost building was necessary to meet budgetary requirements, it was considered advantageous to avoid the perception of low cost because the facility both serves as an international distribution center and has a high degree of local exposure.

The result was a selection of building components that include—in addition to the structural steel framing—prefabricated metal wall panels, single-ply ballasted membrane roofing and precast concrete. The wall panels are metal-faced, insulated sandwich panels that were fabricated off site, installed rapidly and easily maintained. Precast concrete panels were used close to ground level at perimeter exterior walls where potential for damage is greater. Consideration of using pre-engineered metal buildings was precluded by the requirements of fast-track construction, undetermined loading requirements for future conveyors and mezzanines and the need for unusual durability.

A Team Approach to Design and Construction

A fast-track approach requires careful coordination among all involved firms, as well as a mutual respect for the needs of the various parties. The project began with—and was enhanced by—predesign meetings between Purolator Courier, Lockwood Greene and Geupel DeMars, Inc. (the construction manager), during which time *all* major design criteria were established. After this point, significant variations from the agreed-upon criteria would have resulted in cost increases and lengthening the overall project schedule, neither of which occurred.

Additional coordination was required between the architect and the major contractors. Structural loadings, for example, were heavily dependent upon the amount and placement of mechanization equipment. And for this project, the steel was fabricated and erected prior to the commencement of detailed mechanization design. Consequently, *Line haul building during erection.*

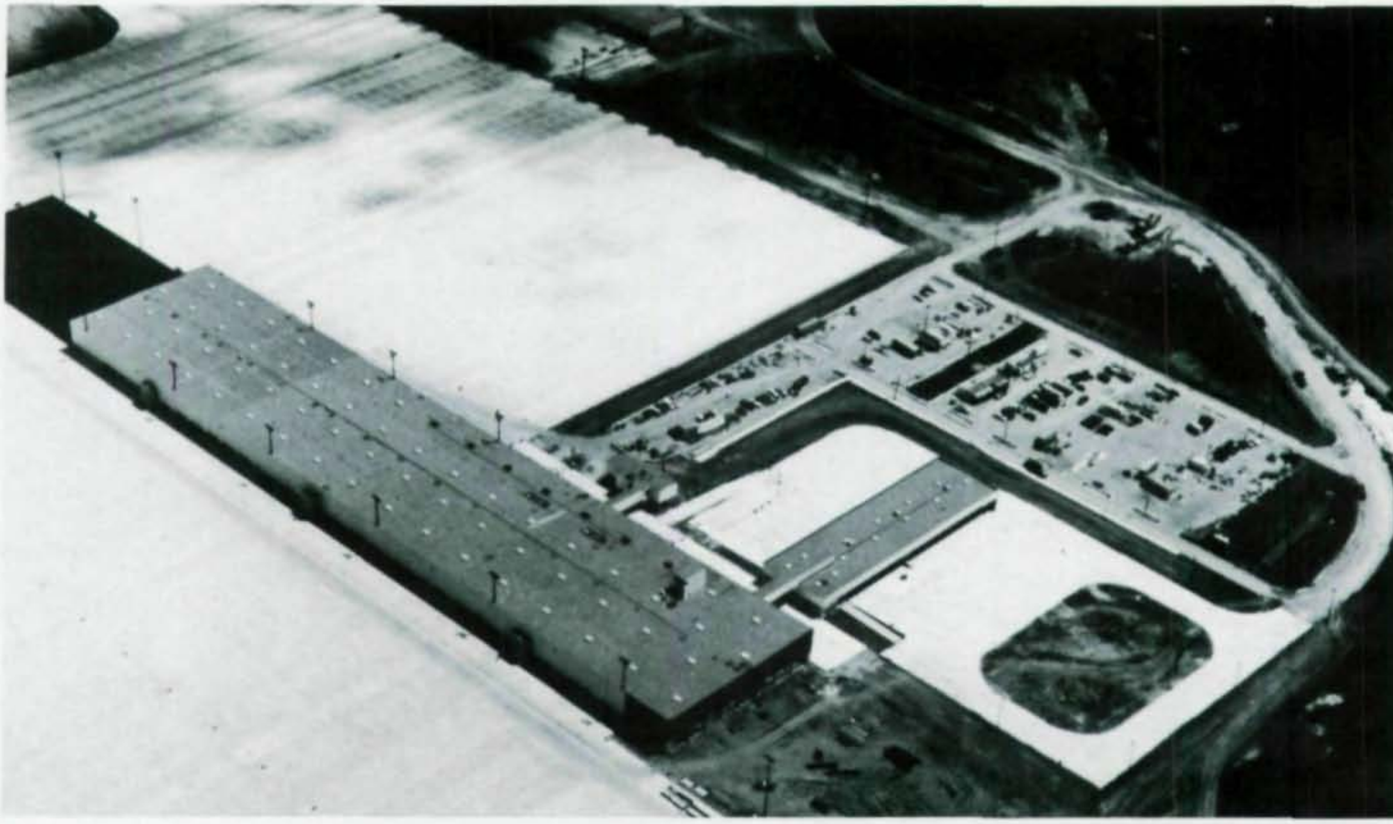
structural engineers spent a considerable amount of time with BAE Automated Systems during the conceptual design phase of the mechanized systems.

The use of structural computer analysis programs (specifically STAAD III and PC software) hastened the design process, and the use of Intergraph CAD systems permitted drawings to be produced at an accelerated rate.

Close in-house coordination among the various design disciplines enabled structural design to begin prior to commencement of other detailed design, without fear of future interferences. This was due in part to the fact the architects had participated in earlier facility programming and conceptual design at Purolator's headquarters—but it was also the result of the team concept on the project.

Plus, success of the fast-track approach can be attributed in a great degree to the use of a construction manager who coordinated the work of subcontractors in such a way as

(continued)



Aerial view defines immensity of steel-framed superhub.

Connections under metal decking

to allow the project to proceed through design and construction without interference from any of the various trades.

Conclusion

A cooperative owner, cooperation on the part of all involved and the selection of flexible, economic materials of construction enabled the completion of the Purolator facility under budget and ahead of schedule. Without a doubt, using steel proved a significant factor in the overall success of the fast-track approach! □

Architect/Engineer
Lockwood Greene Engineers, Inc.
Oak Ridge, Tennessee

Construction Manager
Geupel DeMars, Inc.
Indianapolis, Indiana

Steel Fabricator
Geiger & Peters, Inc.
Indianapolis, Indiana

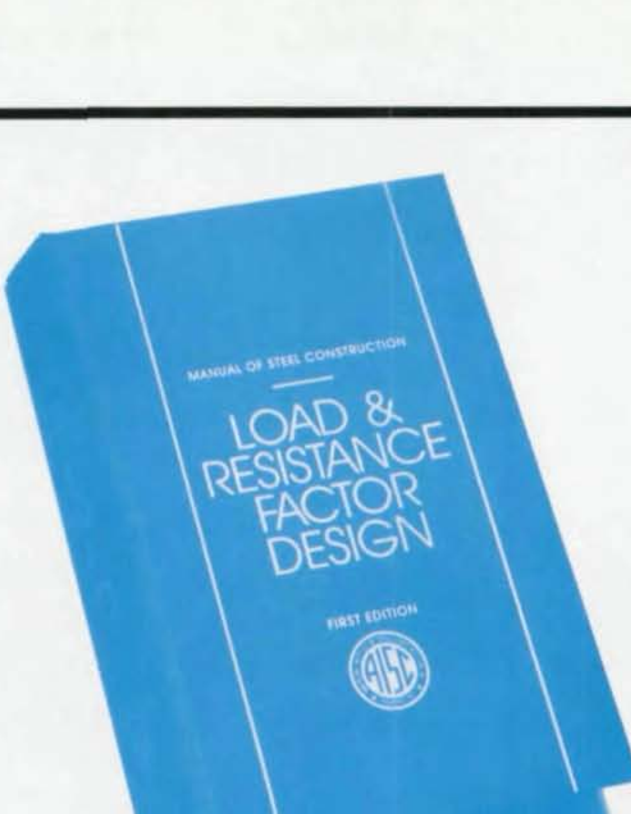
Materials Handling Systems Designer
BAE Automated Systems, Inc.
Dallas, Texas

Owner
Purolator Courier Corp.
Basking Ridge, New Jersey

Edward H. Mahoney, Jr., P.E., is a project engineer for the Oak Ridge, Tennessee, office of Lockwood Greene.

Why LRFD?

by Lynn S. Beedle



Many articles have been written on load and resistance factor design (the American designation for the "limit states design" concept that is a part of the steel design specifications of Canada and of most of the countries of Europe). These articles have dealt with much of the basis for the method, and here and there statements about the advantages of the method have been indicated.

This article summarizes the advantages of the method.

1. LRFD is another tool in the kit of the structural engineer which reflects up-to-date research advances. As a matter of fact, if AISC did not adopt LRFD then it would put design in steel at a potential disadvantage compared to design in other materials. Why not give the steel designers the same tools available to the designer in concrete? Or maybe even better tools?
2. Related to item one is the advantage of flexibility of options. It is *not* intended that use of LRFD be mandatory. It will be an alternate. What happens later depends a lot upon the marketplace. Eventually, I think we should have only LRFD, which will encompass both ASD and PD.
3. Allowable-stress design is an approximate way to account for what LRFD does in a more rational way. The introduction of plastic design concepts resulted in so many changes in the Part 1 section of the Allowable Stress Specification that it can certainly no longer be called an "elastic design" process. All the same, LRFD does this much more rationally with its clear call in the design process for decisions on the loading condition and the response condition.
4. The rationality of LRFD has always been attractive. Normally this appeal is not a strong factor in the decision as to whether or not a new design technique will be adopted. But in the case of LRFD it is a powerful incentive. On the one side its rationality means that better use can be made of material, which will inevitably lead to savings for some load combinations and structural configurations. On the other hand, it will lead to safer structures in view of the present admittedly arbitrary uniformity of considering all possible load distribution systems as the same.
5. The use of multiple load factors, instead of the present limited selection of two, should lead to economy. After all, if one uses different factors for live load and dead load, then there are frequent situations in which economy is inevitable.
6. The method will facilitate the input of new information on loads and load variations when that information becomes available. We know a fair amount about the resistance of steel structures. In comparison, our knowledge of loads and their variation is much less. Some research is underway in this field, but much more is needed. When the new information does become available, a Specification that is already prepared in the LRFD format will be one in which changes can be rapidly incorporated. Since the loading function is separated from the resistance function in LRFD, one can be changed without a reexamination of the other.
7. The same thing is true with regard to resistance factors. Not all of the answers are available about the behavior of steel structures—especially in the field of stability and connections. When new information on resistance becomes available it can be more readily incorporated.
8. In connection with items 6 and 7, in all likelihood changes in load factors and resistance factors will be easier to make than carte blanche changes in allowable stress. Changing the allowable stress appears to be a more radical revision than a change in a load factor or in a resistance factor.
9. LRFD makes design in all materials more compatible. The need for this has frequently been stated by designers who work with all materials—not just wood or

steel or concrete. So a specification that is not written in the limit-states format will be at a disadvantage—at least with respect to the material represented by that specification.

10. LRFD provides a framework to handle unusual loads that may not be covered by the Specification. When the designer encounters an unusual situation it may well be his uncertainty has to do with the resistance of the structure. In this case, he can modify the resistance factor. In other cases, his uncertainty may have to do with the loading. In that case he can conveniently modify the load factor without tampering at all with the resistance factor for the structure or element.

11. Another advantage has to do with new materials. We must not suppose the last word has been written and the last invention made which affects the characteristics of the materials being used. When a new material is introduced, once the statistical information is determined about properties, and the role of that variation is understood as far as members and structures are concerned, a new resistance function can be introduced. Again, this will not affect the loading function, so the change can be made by simple substitution.

12. Another advantage has to do with calibration. Quite unlike plastic design, where calibration was carried out on a structural element (simple beam) which had a long record of satisfactory performance—but which had the *least* inherent reserve strength—calibration in LRFD was carried out on an *average* situation. This calibration could well change in the future. When such a change occurs, the LRFD format lends itself to a not-too-complicated transition to accommodate the new information. (Also, this would appear to improve the weight-saving aspect of LRFD.)

13. Some live-load/dead-load ratios will lead to diseconomies, especially when the dead load is a low percentage. On the other hand, there will be cases where, under high dead load, there will be economies in LRFD insofar as weight savings are concerned.

14. We could actually have safer structures under LRFD than at present because the method should lead to a better awareness of what is going on. (The people who have studied these things tell us our design approach is really quite crude—but it is a tribute to the vision, art and experience of practicing engineers that it works as well as it does.) Turkstra emphasized this point in *Civil Engineering* (June 1982, p. 68). Of course there are a host of other references that speak to the same point.

15. We are only at the beginning when it comes to serviceability limit states. In present design we are at sea. In LRFD at least we have an approach.

We come back to the original theme. Most design practice can proceed in the future as it has in the past, usually on an allowable-stress design basis and in some instances by making use of plastic design. On the other hand, with the addition of LRFD, it becomes possible for the designer to move in a safer and more economic way into other areas. And the inevitable thrust will be to move gradually more and more in the direction of LRFD. It is here that the best advantage can be taken in design of knowledge about the behavior of materials, elements and structures and how most loads act on most structures. And LRFD is about the only way a specification can help the designer deal with special loads on special structures.

Lynn S. Beedle is Professor of Civil Engineering and Director, Fritz Engineering Laboratory, Lehigh University, Bethlehem, Pennsylvania

AISC Prize Bridge Awards 1986

The 1986 Prize Bridge and Merit Award winners have been announced. This year, a total of 16 winners in both categories were chosen as the most handsome and functional bridges opened to traffic between July 1, 1984 and June 30, 1986. Eight Prize Bridge Awards were selected in the 10 judging categories. Also, eight Award of Merit winners were chosen for recognition.

The awards will be presented to the designers of the winning Prize Bridge Award structures at the prestigious 6th Annual Awards Banquet in Chicago's Westin Hotel on Dec. 2. Leaders of the construction industry gather to recognize these designers. Plaques adapted from the Joe Kinkel-designed sculpture, "The Long Reach," will be presented to the winners by AISC Chairman Werner Quasebarth.

The Prize Bridge Competition, conducted since 1928, has inspired much greater attention to the aesthetics of bridge design as well as to the advancement of steel as a structural material. The winners are. . .

Prize Bridge Competition jurors (l. to r.) Robert D. Bay, Frank D. Sears, Louis A. Garrido and James E. Sawyer chose top bridges from a wide field of entries.

THE JURY OF AWARDS

ROBERT D. BAY
Engineering Manager, Black & Veatch
Kansas City, Missouri and
President, ASCE
New York, New York

LOUIS A. GARRIDO
Bridge Design Engineer,
Louisiana DOT
Baton Rouge, Louisiana

JAMES E. SAWYER
Chief Operating Officer,
Greiner Engineering, Inc.
Tampa, Florida

FRANK D. SEARS
Chief/Review and Analysis Branch, Federal Highway Administration
Washington, D.C.

STANLEY TIGERMAN
President, Tigerman Fugman & McCurry
Chicago, Illinois and
Director, School of Architecture,
University of Illinois
Chicago, Illinois

Winners receive a bas relief of a single-edition bronze sculpture by Joe Kinkel. Sculpture and engraved names of winners are on display at AISC's Chicago headquarters.

PRIZE BRIDGE 1986—
GRADE SEPARATION 1

MELROSE INTERCHANGE BRIDGE

Nashville, Tennessee

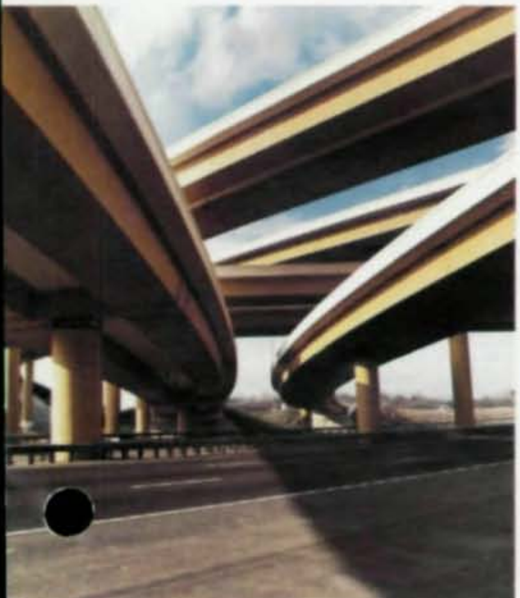
This major interchange of two Interstate Highways was built within a confined area bounded by a four-lane primary route, a double-track rail line and a dense commercial district. The horizontally curved, constant depth, steel plate box girders give it its clean lines. Fully continuous from abutment to abutment, each structure has twin boxes, varying in length from 462 to 705 ft, connected at bents to integral steel box caps. With a maximum span length of 281 ft, the structure has a support system of a framed-in cross girder and a single-column arrangement, which adds to the structure's attractiveness.

Designer
Division of Structures, Tennessee DOT
Nashville, Tennessee

General Contractor
The E. Randle Company
Frankford, Kentucky

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

Owner
Tennessee DOT
Nashville, Tennessee



1

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 long, or more

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

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

PRIZE BRIDGE 1986—
RECONSTRUCTED 2

GOLDEN GATE BRIDGE-DECK REPLACEMENT

San Francisco, California

Jurors appreciated the excellent use of an orthotropic-type replacement deck on a bridge of this magnitude. The method offered cost effectiveness and weight benefits. It is about 30% lighter than the original roadway. In addition, the replacement decks were immediately usable on installation, compatible with the existing deck at its interface, prefabricated and modular, allowing the bridge to be kept open to serve San Francisco Bay area commuters. The project covered a length of 8,981 ft and encompassed a deck area of 566,000 sq ft, requiring 756 individual erection units.

Designer
Ammann & Whitney
New York, New York

General Contractor/Steel Erector
Dillingham/Tokola
Pleasanton, California

Steel Fabricator
Chicago Bridge & Iron Company
Salt Lake City, Utah

Owner
Golden Gate Bridge, Highway and Transportation District
San Francisco, California

2

PRIZE BRIDGE 1986—
LONG SPAN 3

I-255 OVER MISSISSIPPI RIVER

Jefferson Barracks, Missouri

"Graceful" tied-arch bridge, with a 909-ft span between tie points, is thought to be the longest of its kind in the U.S. The tie is an I-shape instead of box, which required less material and eased fabrication. Jurors found the Vierendeel bracing of the arch particularly open, free, clean and very elegant looking. Innovative design called for prestressing of the arch rib and tie so there were no flexural stresses in either member.

Designer
Alfred Benesch & Company
Chicago, Illinois

Steel Fabricator/Erector
American Bridge, Div. of U.S. Diversified Group, U.S.X. Corp.
Pittsburgh, Pennsylvania

Owners
Illinois DOT
Springfield, Illinois and
Missouri Highway and Transportation Department
Jefferson City, Missouri

PRIZE BRIDGE 1986—
RAILROAD 4

VISTA AVENUE RAILROAD BRIDGE

Boise, Idaho

Simple span arrangement and graceful treatment of the abutments, along with the framing, developed a very pleasing railroad structure. The brown tones of the A588 steel used for the 104-ft long bridge blend with the surrounding trees. Use of trapezoidal steel box girders to create a through-girder arrangement is unique. The exterior webs are sloped, with a smaller bottom flange than top flange.

Designer
CH2M Hill, Inc.
Boise, Idaho

General Contractor
Harcon Incorporated
Pocatello, Idaho

Steel Fabricator
Morrison-Knudsen Company, Inc.
Boise, Idaho

Steel Erector
Sorenson Steel Inc.
Idaho Falls, Idaho

PRIZE BRIDGE 1986—
MOVABLE SPAN 5

WELLS STREET VERTICAL LIFT BRIDGE

Milwaukee, Wisconsin

A vertical lift bridge that does not look like one because it does not have the usual extended towers. The lift span resembles a rectangular, four-legged table. When closed, the four legs rest within two hollow piers that extend below the water line. The counterweight is located within the pier between these two legs. The equalizer system is strung diagonally across the lift span rather than longitudinally and transversely.

Designer
Department of Public Works, City of Milwaukee

General Contractor/Steel Erector
Edward Kraemer and Sons, Inc.
Plain, Wisconsin

Steel Fabricator
Theo. Kupfer Iron Works, Inc.
Monona, Wisconsin

Owner
City of Milwaukee

PRIZE BRIDGE 1986—
ELEVATED HIGHWAY/
VIADUCT 6

SIXTH STREET BRIDGE OVER JEFFERSON AVENUE

Detroit, Michigan

Clean lines of the blue-painted structure, which blends well with surrounding sky and water, attracted jurors' attention. The bridge's live load is HS 25, with ASTM A-572, Gr. 50 steel, used throughout and an alternate option of A-307 for anchor bolts. The substructure has cantilever arms on the piers, but they do not present a massive appearance.

Designer
Madison Madison International of Michigan
Detroit, Michigan

Consultant (foundations)
Sidney E. Shorter & Associates
Detroit, Michigan

General Contractor/Steel Erector
Barton-Malow Co.
Detroit, Michigan

Steel Fabricator
Phoenix Steel, Inc.
Eau Claire, Wisconsin


3


4


5


6

PRIZE BRIDGE—1986 7
MEDIUM SPAN, LOW CLEARANCE

BECTON DICKINSON ACCESS BRIDGES (EAST & WEST)

Franklin Lakes, New Jersey

Each bridge has a single span which provides little interruption to the visual continuity of the roadway. Placed between natural embankments well covered with trees, the bridges penetrate the landscape and rest on solid, but minimal, abutments. Design rendered the plate girder elements fracture critical, and design and fabrication were adopted to meet the technical requirements. Treated with a zinc-rich primer, epoxy coated and given a urethane finish coat, the plate girders' color mixes well with the natural setting.

Co-designers
Zaldastani Associates, Inc.
Boston, Massachusetts and
Kallmann, McKinnell & Wood
Boston, Massachusetts

General Contractor/Steel Erector
R.A. Hamilton Corporation
Hackensack, New Jersey

Steel Fabricator
Lehigh Structural Steel Company
Allentown, Pennsylvania

Construction Manager
Gilbane Building Co.
Princeton, New Jersey

Owner
Becton Dickinson Real Estate Co. Inc.
Franklin Lakes, New Jersey


7

PRIZE BRIDGE 1986—
MEDIUM SPAN, HIGH
CLEARANCE 8

McCALLUM STREET OVER CRESHEIM CREEK

Philadelphia, Pennsylvania

The bridge was designed to provide park land in the valley beneath. Jurors commended unique solutions of fabrication and erection problems. The bridge has three frames spaced 16 ft-6 in. o.c. with W21×62 rolled sections spanning between them. Span lengths of the frame are 152 ft-6 in. for the end spans and 235 ft for the middle span. For the design of the upper part of the frame, the effective span lengths realized are 93 ft-9 in. for the end spans and 117 ft-6 in. for the three interior ones.

Designer
Bridge Section, Philadelphia Streets Department
Philadelphia, Pennsylvania

General Contractor
The Nyleve Company
Emmaus, Pennsylvania

Steel Fabricator
High Steel Structures, Inc.
Lancaster, Pennsylvania

Steel Erector
Lindstrom & Companies, Inc.
Cinnaminson, New Jersey

Owner
City of Philadelphia
Philadelphia, Pennsylvania

AWARD OF MERIT 1986— 9
MEDIUM SPAN, LOW CLEARANCE

FALLS ROAD BRIDGE

Village of Grafton, Wisconsin

Designer/Consulting Firm
Donohue & Associates, Inc.
Madison, Wisconsin

General Contractor
Lunda Construction Company
Black River Falls, Wisconsin

Steel Fabricator
Phoenix Steel, Inc.
Eau Claire, Wisconsin

Steel Erector
Hi-Boom Erecting, Inc.
Black River Falls, Wisconsin

Owner
Village of Grafton

AWARD OF MERIT 1986—
ELEVATED HIGHWAY/
VIADUCT 10

MERRITT PARKWAY E-S & W-N ROADWAYS

Trumbull, Connecticut

Designer
Seelye Stevenson Value & Knecht
Stratford, Connecticut

General Contractor
Arute Brothers, Inc.
New Britian, Connecticut

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

Owner
State of Connecticut
Wethersfield, Connecticut

AWARD OF MERIT 1986—
RECONSTRUCTED 11

MARY HUNNEWELL FYFFE FOOTBRIDGE

Newton-Wellesley, Massachusetts

Designer
Barnes and Jarnis, Inc.
Boston, Massachusetts

General Contractor/Steel Fabricator/Steel Erector
Engineering Construction, Inc.
Ipswich, Massachusetts

Owner
Metropolitan District Commission
Boston, Massachusetts

AWARD OF MERIT 1986—
SHORT SPAN 12

PACIFIC STREET BRIDGE

Appleton, Wisconsin

Designer
Ayres Associates
Eau Claire, Wisconsin

General Contractor
Lunda Construction Company
Black River Falls, Wisconsin

Steel Fabricator
Hartwig Manufacturing Corporation
Wausau, Wisconsin

Steel Erector
Hi-Boom Erecting, Inc.
Black River Falls, Wisconsin

Owner
City of Appleton

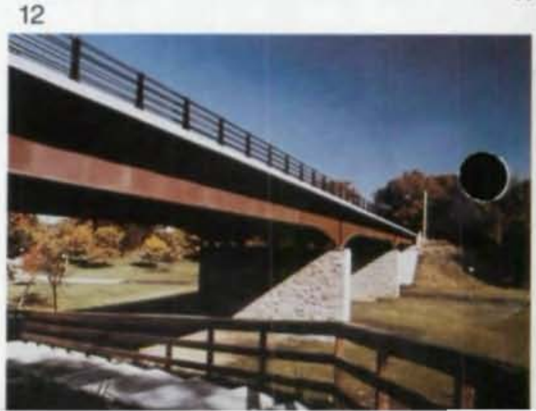
8

9

10

11

12



13

14

15

16

AWARD OF MERIT 1986—
SPECIAL PURPOSE 13

6TH STREET MARKETPLACE
Richmond, Virginia

Design Architect
Wallace, Roberts & Todd Architects
Philadelphia, Pennsylvania

Consulting Architect
Marcellus Wright, Cox & Smith
Richmond, Virginia

Structural Engineers
Harris, Norman & Giles Consulting Engineers
Richmond, Virginia and
Jackson & Tull
Washington, D.C.

Construction Manager
The Whiting-Turner Contracting Company
Richmond, Virginia

Steel Fabricator
Liphart Steel Company, Inc.
Richmond, Virginia

Steel Erector
Wall's Welding Service
Glen Allen, Virginia

Owners
Richmond Festival Marketplace Partnership
Richmond, Virginia and
The Enterprise Development Company
Columbia, Maryland

AWARD OF MERIT 1986—
SPECIAL PURPOSE 14

SKYRIDE at MIAMI INTERNATIONAL AIRPORT
Miami, Florida

Designer
Bliss & Nyitray, Inc.
Miami, Florida

Architect
M. C. Harry & Assoc. Inc.
Miami, Florida

General Contractor
Darin & Armstrong, Inc. (subsidiary of Walbridge Aldinger Co.)
Livonia, Michigan

Steel Fabricator
Tampa Steel Erecting Company
Tampa, Florida

Steel Erector
Dixie Metal Products, Inc.
Fort Lauderdale, Florida

Owner
Aviation Department, Metropolitan Dade County
Miami, Florida

AWARD OF MERIT 1986— 15
MEDIUM SPAN, LOW CLEARANCE

BONNERS FERRY BRIDGE
Bonners Ferry, Idaho

Designer
T.Y. Lin International
San Francisco, California

General Contractor/Steel Erector
Kiewit Pacific Company
Vancouver, Washington

Steel Fabricator
Robberson Steel & Bridge Company
Oklahoma City, Oklahoma

Owner
Idaho Transportation Department, State of Idaho
Boise, Idaho

AWARD OF MERIT 1986—
SHORT SPAN 16

B & SOMERSET STREETS OVER CONRAIL
Philadelphia, Pennsylvania

Designer
Bridge Section, Philadelphia Streets Dept.
Philadelphia, Pennsylvania

General Contractor
Belbold Contracting Corporation
Bristol, Pennsylvania

Steel Fabricator
Delsea Parker Corporation
Millville, New Jersey

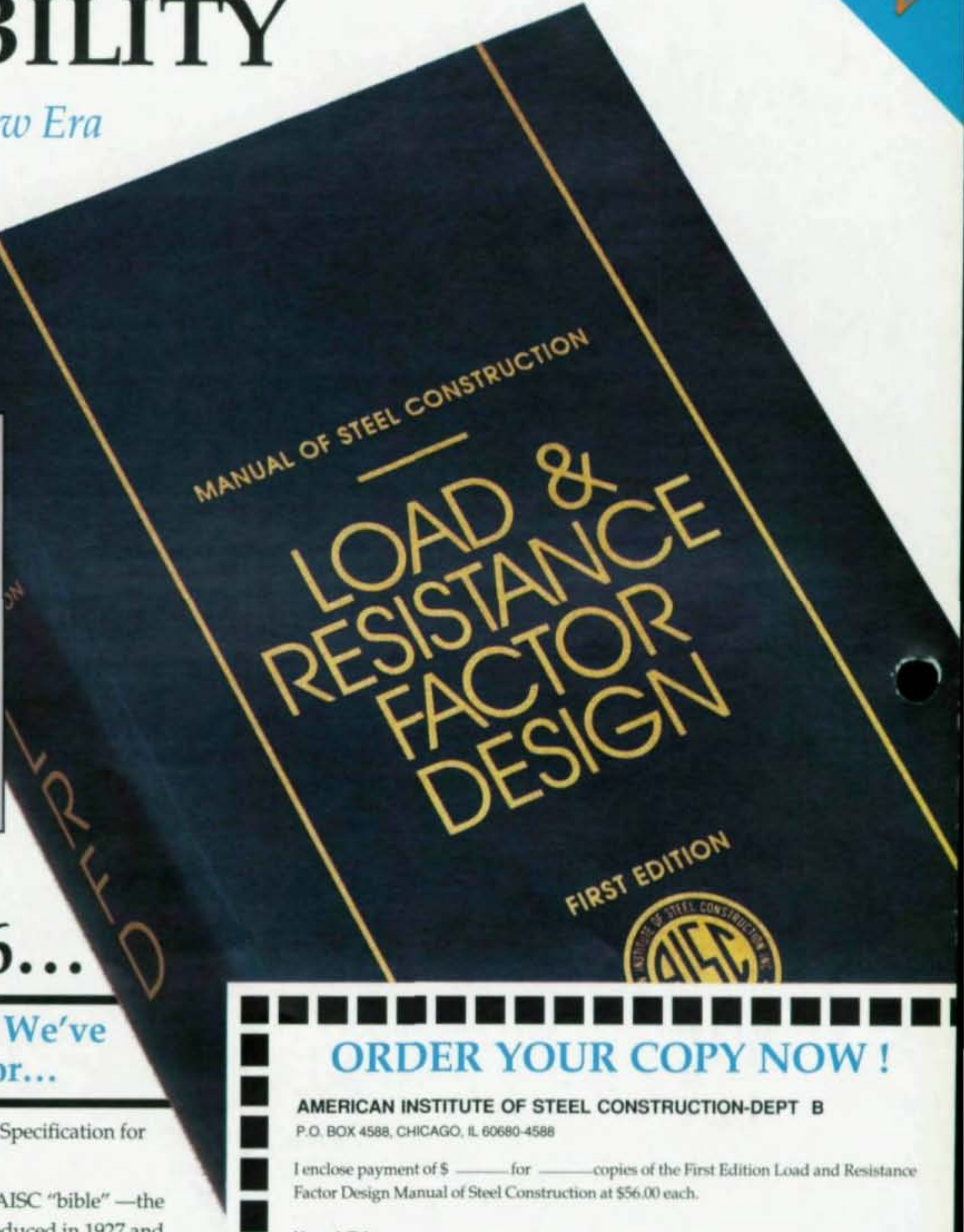
Steel Erector
Coastal Steel Construction Company, Inc.
Glenside, Pennsylvania

Owner
Philadelphia Streets Department
Philadelphia, Pennsylvania

CONTINUING A TRADITION OF RELIABILITY

1ST EDITION

...Launching A New Era in Steel Design!



1927...

1986...

It's Here! The Book We've All Been Waiting For...

- based on the 1986 AISC "LRFD" Specification for Structural Steel Buildings.
- grounded in the tradition of the AISC "bible" —the *Manual of Steel Construction*, introduced in 1927 and now in its 8th Edition.
- meshing the familiar and accepted basics with the very latest approaches to steel design, derived from current research and technological advances.
- helping you design more reliable, more economical steel-framed buildings.

ORDER YOUR COPY NOW !

AMERICAN INSTITUTE OF STEEL CONSTRUCTION-DEPT B

P.O. BOX 4588, CHICAGO, IL 60680-4588

I enclose payment of $ _____ for _____ copies of the First Edition Load and Resistance Factor Design Manual of Steel Construction at $56.00 each.

Name & Title __________

Company __________

Address __________

City __________ State _____ Zip _____

Please enclose remittance. No C.O.D. orders. In New York, California and Illinois add sales tax. Shipping charges prepaid in the U.S. On shipments outside the U.S., add 10% of total purchase for postage and handling. **Visa and MasterCard accepted.**

Charge My Card No. __________ **Exp.** _____