# MODERN STEEL CONSTRUCTION

November 1994

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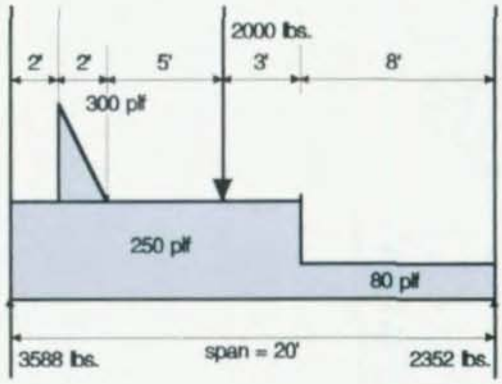
In the past it was necessary to show special loadings, such as this example, on the structural drawings and call for a special (SP) joist.

example




maximum moment = 243 k. inch
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Now, tables for new KCS joists can be used to find a joist with sufficient moment and shear values. The new KCS joists have constant moment and shear strengths -- in the case of the illustrated example a 16KCS2 joist is found to resist a moment of 349 inch kips and a shear of 4000 pounds. By specifying this joist it is no longer necessary to call for a SP joist or show the diagram.

In addition to the tables on the new K Series KCS joists the new SJI specification contains other important revisions:

1. New bridging requirements for K, LH, and DLH joists.
2. Metric load tables.
3. Revised joist girder bearing seat depths.

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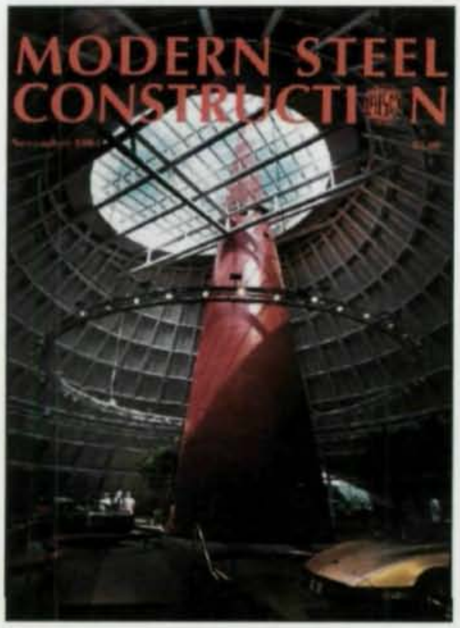
**Please circle # 33**

# MODERN STEEL CONSTRUCTION

Volume 34, Number 11 November 1994




*Bold exhibition spaces should help attract a crowd of Corvette enthusiasts. The story of the exciting museum begins on page 24.*

## FEATURES



## DEPARTMENTS



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# Building Boom

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When AISC first moved to its new offices in 1989, I had a great view from my 31st floor office. Since then, two high rises have gone up—substantially obscuring my once-grand vista. Oh, the many hours I've daydreamed about blowing up one of the offending structures. Apparently, I'm not the only one with these daydreams.

The *Chicago Reader*, an "alternative" free weekly, recently asked prominent Chicago architects about buildings they'd like to see blown up (though, ostensibly, the reasons were supposed to be for aesthetic value not mere personal pique). Since most in the building industry are familiar with Chicago's skyline, I thought I'd share with you some of these architect's top choices. Remember, the sampling was decidedly unscientific—consisting of only 29 votes with the top vote-getter receiving only four votes.

The top choice for destruction deeply distressed me, because it's my wife-to-be's favorite Chicago building: 150 North Michigan Ave. (better known as the Associates Building and best known for its diamond-shaped roofline that is so prominent in skyline photos and in such movies as the "Adventures in Babysitting") was decried because it's too noticeable and doesn't pay homage to Chicago's architectural traditions. I was a lot happier about the fourth building on the list: R.R. Donnelly Center at 77 W. Wacker. Besides being unattractive, this is one of the buildings that blocks my view.

Perhaps the most expected building on the list is number 14, the James R. Thompson Center (Helmut Jahn's curving "spaceship"). This building was exceedingly controversial when built—most people hated it, some loved it, no one was neutral. Personally, I always loved it visually, though I've had friends who worked in the building who were miserable due to the horrifying acoustics created by the full-height, open atrium in the building's center.

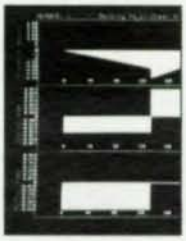
The most unexpected building was number 19, Mies van der Rohe's brilliant IBM Center at 330 N. Wabash. However, it wasn't cited for any fault of its own, but as a precursor of numerous bad Mies knockoffs scattered haphazardly around the city.

On the more serious side, some of the architects did make relevant points about city planning. One designer chided the Stouffer Riviere Hotel (across the street from my office but too low to affect my view) because while it is sited on a bend in the Chicago River, the guest rooms are set back, effectively nullifying one of the best vantage points in the city.

The Museum of Contemporary Art was criticized even before its completion for similar reasons. Its squat form will block one of the last potentially open views of Lake Michigan from the shopping mecca of Michigan Avenue. The Harold Washington Library Center was blasted for its overblown details (the owls on top are oversized and pedestrians sometime cross the other side of the street out of fear they'll fall off the building) and its lack of user-friendliness; quite simply, it is not an inviting or easy-to-use structure. And finally, the Robert Taylor Homes were decried as being misplanned social policy—something I doubt anyone would disagree with.

If you want a complete list of architect's least favorite Chicago building's, you'll have to get it from the *Reader*. However, if you want to add to the list—or nominate a building in another city, jot me a note. I always like hearing from MSC readers. **SM**

# How to get from here



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# AISC Design Guides


AISC has published a series of seven Design Guides to supplement the *Manual of Steel Construction*. To order any of the publications listed below, call **312/670-2400**.

***•D801–Column Base Plates:*** Contains a compilation of existing information on the design of base plates for steel columns and anchor bolts in building frames. Design guidelines are included. ($20.00)

***•D802–Design of Steel and Composite Beams with Web Openings:*** Summarizes design concepts for the practicing engineer and presents a unified design approach to both steel and composite beams with web openings, including reinforcement requirements. ($20.00) Companion computer program is available.

***•D803–Serviceability Design Considerations for Low-Rise Buildings:*** Summarizes existing research and criteria and presents requirements for deflections, vibration, drift and other serviceability limits. ($16.00)

***•D804–Extended End Plate Moment Connections:*** Covers the design, fabrication and erection of extended end-plate connections and gives recommendations for four-bolt and eight-bolt end-plate connections. ($20.00)

***•D805–Design of Low- and Medium-Rise Steel Buildings:*** Consolidates a vast quantity of information on this subject into one booklet. Also includes rules for economic design for engineers, loading requirements, and joist and composite floor systems. ($20.00)

***•D806–Load and Resistance Factor Design of W-Shapes Encased in Concrete:*** Covers design of composite columns comprised of rolled wide-flange shapes encased in structural concrete with vertical deformed reinforcing bars. The advantages and disadvantages of composite construction, as well as practical design suggestions, are covered. ($20.00) Companion computer disk also is available.

***•D807–Industrial Buildings: Roofs to Column Anchorage:*** Covers industrial buildings both with and without cranes. Information is provided on load conditions and combinations, roof systems, framing systems, wall systems, column design and column anchorage. ($30.00)

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# STEEL INTERCHANGE

*Steel Interchange* is an open forum for *Modern Steel Construction* readers to exchange useful and practical professional ideas and information on all phases of steel building and bridge construction. Opinions and suggestions are welcome on any subject covered in this magazine. If you have a question or problem that your fellow readers might help you to solve, please forward it to *Modern Steel Construction*. At the same time, feel free to respond to any of the questions that you have read here. Please send them to:

**Steel Interchange**
**Modern Steel Construction**
**One East Wacker Dr., Suite 3100**
**Chicago, IL 60601-2001**

Answers and/or questions should be typewritten and double-spaced. Submittals that have been prepared by word-processing are appreciated on computer diskette (either as a Wordperfect file or in ASCII format).

The opinions expressed in *Steel Interchange* do not necessarily represent an official position of the American Institute of Steel Construction, Inc. and have not been reviewed. It is recognized that the design of structures is within the scope and expertise of a competent licensed structural engineer, architect or other licensed professional for the application of principals to a particular structure.

Information on ordering AISC publications mentioned in this article can be obtained by calling AISC at 312/670-2400 ext. 433.

***The following responses from previous Steel Interchange columns have been received:***

**The use of channel sections or other lightweight narrow flange sections as girts supporting non-bearing exterior wall assemblies against wind load is common practice. How is lateral instability of the unsupported compression flange accounted for when the wall is subject to outward pressure due to suction at the leeward face of the building? These outward forces are equal to or greater than the inward forces.**

It is instructive to determine the force required to prevent buckling. A C8 x 11.5 is commonly used as a girt. For simplicity, assume the full flange is stressed to 20 ksi. The force developed by the flange is 17.6 kips (0.081 in.$^2$ x 20 ksi = 17.6 kips). Invoking the 2 percent rule gives a required resisting force to prevent buckling of 350 pounds. This is small compared to the forces structural engineers usually consider.

I note that girts are generally held level with one or more sag rods spaced along the girt's span. The sag rods commonly pass through the girt web and are secured with a single nut placed beneath the girt.

If the girt is installed with a slight bow downward, the girt flange can only laterally buckle downward since it is not possible to reverse initial curvature. Provided the sag rod is placed close enough to the compression flange, any tendency to buckle will be resisted by the sag rod in tension. The relevant question is how close to the unbraced flange must the sag rod be? I do not know of any authoritative guidance on this matter.

However, if the girt is initially set with an upward deflection and supported by a single nut below the web, the girt may laterally displace upward unhindered except by gravity. this problem may be overcome by providing a double nut connection - one below and one bolt above the web. This would place the sag rod in compression. Although the sag rod's capacity in compression is small, the force required to resist flange buckling is also small.

In any event, thousands if not hundreds of thousands of industrial buildings have been constructed with metal siding supported by girts and the failure of girts under wind loading does not appear to be a serious problem. I believe that in many of these buildings little thought was given to the unbraced girder flange. I have personally observed "pre-engineered" buildings where light gage Cee or Zee girts are stayed between columns by the exterior siding only.

I speculate that girt supported walls perform satisfactorily for several reasons, regardless of the potential for flange instability. One, code dictated wind loads are conservative and rarely achieved. When achieved, it is likely that the wall diaphragm is breached relieving some of the loading. Also, wind loads are short-lived and the inertia of the girt-wall system results in a time-lag before the girt is fully deflected and stressed. During that delay the gusting wind may slow or change direction. Should the girt flange buckle, additional load carrying capacity may be developed through catenary action. In other words, the girt becomes a tension member restrained by its supports. And of course, the girt to column connection may afford substantial continuity.

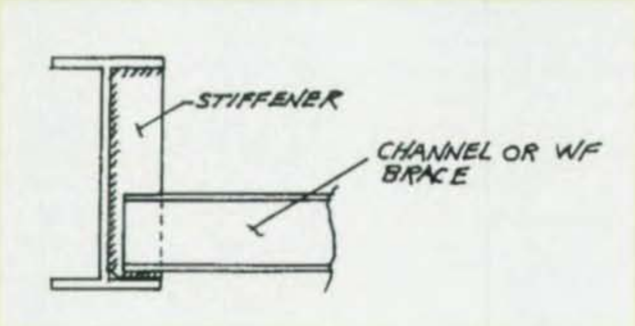
***Robert Busch***
**Leonard Busch Associates**
**Trenton, NJ**

**When erecting steel beams on a brick wall, could the non-shrink grout be omitted under a proper bearing plate if the surface of the brick is smooth, clean of any and all debris and leveled?**

Even though the author of the question describes the brick bearing surfaceas "smooth, clean...., and leveled", the degree of perfection commonly found in brick masonry is no match for the flatness of a steel bearing plate.

Without grout the load will not be uniform but rather will be concentrated at the high points of the brick. This may lead to fragmentation of the brick masonry. One of the purposes of grout is to help distribute the load uniformily. It should not be omitted.
*David Ricker, P.E.*
**Payson, AZ**

**What type of framing is considered bracing the compression flange? Does the member bracing the flange have to be attached to the flange? If a 4 inch deep member frames into mid-depth of a 10 inch deep beam is that considered bracing the compression flange (centerlines of each member at same point)?**




As discussed in the ASD 9th Edition Section C-F1, strong-axis-bending lateral-buckling can be prevented either by bracing the compression flange directly or by preventing the section from twisting. The first of these methods can be done using a concrete slab, properly attached deck or steel framing. Due to coping or framing considerations, steel braces often are not directly attached to the compression flange. A rule of thumb is that the connection for a brace should extend at least into the upper $1/3$ of the beam depth to consider it as a lateral support for the top compression flange. Where this is not possible, the detail shown could be used to assure lateral support.
*Kenneth Wislocky, P.E.*
***Raytheon Engineers & Constructors***
**Philadelphia, PA**

**Can threads on anchor bolts be either rolled or cut? Is one method better than the other?**

Anchor bolt threads may be cut or rolled, depending on the project specification requirements. Allowable stresses published in AISC and other codes account for any differences in strength which may result from one method or the other.

A good article on this subject is titled, "Rolled Threads vs. Cut Threads," dated June 20, 1966. This article was published in Fastener Facts and was released in the mid 1970's by the Bowman Products Division of the Associated Spring Corp. of Cleveland Ohio. To summarize, the article states that rolling and deforming the grain structure, rather than cutting through it , results in additional strength or resistance to thread shear.
*Dennis D. Havranek*
**Valmont Industries, Inc.**
**Valley, NE**

## New Questions

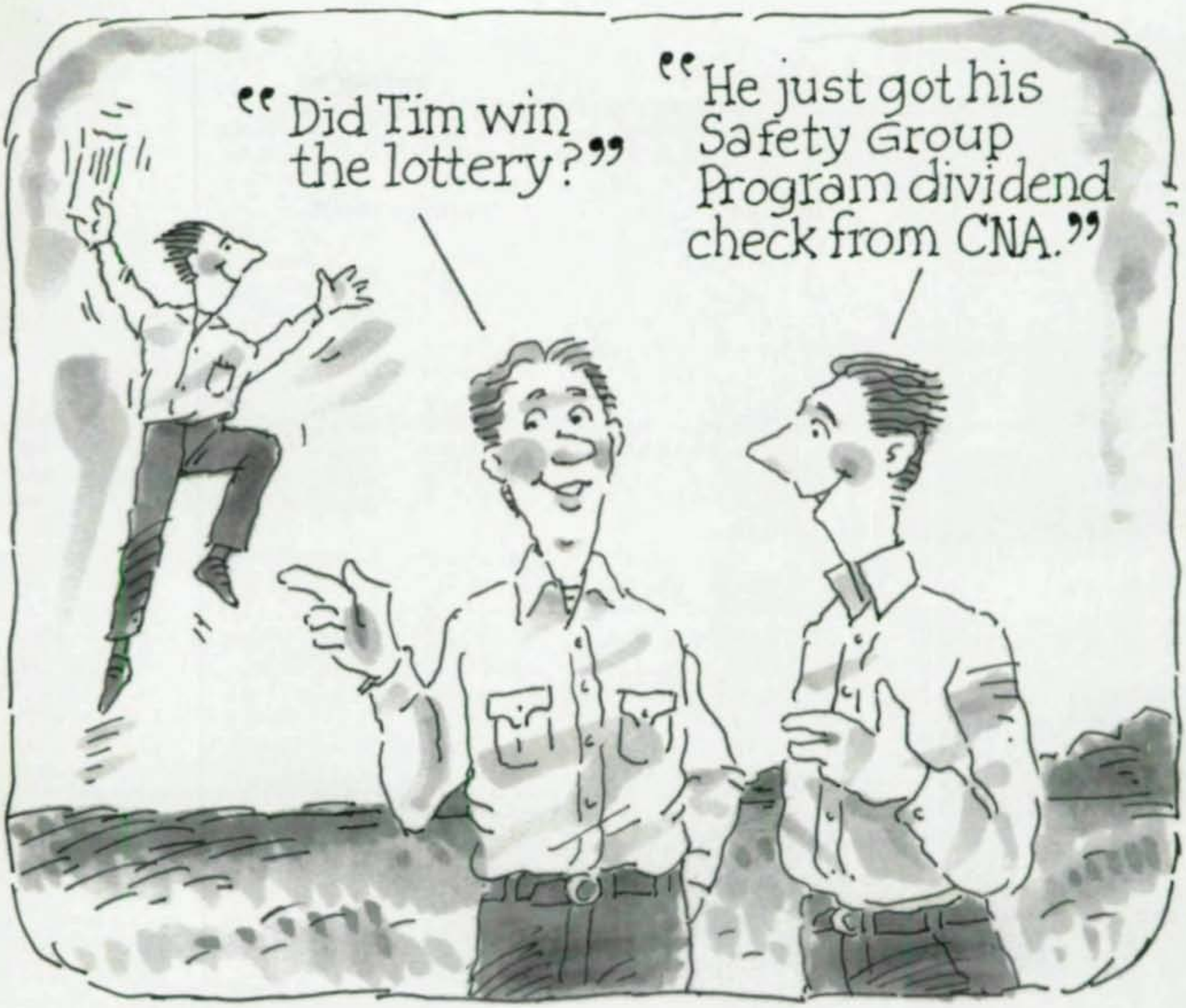
Listed below are questions that we would like the readers to answer or discuss.

If you have an answer or suggestion please send it to the Steel Interchange Editor, Modern Steel Co nstruction, One East Wacker Dr., Suite 3100, Chic ago, IL 60601-2001.

Questions and responses will be printed in future editions of Steel Interchange. Also, if you have a question or problem that readers might help solve, send these to the Steel Interchange Editor.

In regard to single plate shear connections, Table X of allowable loads in ASD 9th Ed. is based on conservative assumptions such that in many cases the bolt diameter, type, or quantity must be increased to satisfy the required loads. Is the method used in Engineering for Steel Construction (University of Arizona research) still acceptable, particularly when the job specifications call for using the "latest AISC standards"?
*Aaron Snyder*
**Leonard/Mercurio & Associates**
**Pittsburgh, PA**

Specifications currently exist which require minimum pretensioning loads for slip critical connections. There is, however, no guidance regarding minimum pre-loading of anchor bolts which occur at column bases. While in most situations this issue is academic since the anchor bolt nut and thread projection are below the plane of the concrete slab on grade and are eventually embedded in concrete at the slab isolation joint, there are instances where the nut and thread projection remain exposed. Is tightening the nut to "snug tight" and tack welding the nut to the bolt thread the only solution in preventing the nut from backing off?
***Charles F. Canitz, P.E.***
**U.S. Army Corps of Engineers**
**Baltimore, MD**


"Did Tim win the lottery?"
"He just got his Safety Group Program dividend check from CNA."

# PEDDINGHAUS SHOWS STATE-OF-THE-ART FABRICATION

Peddinghaus' biennial Oktoberfest open house (September 26 - October 7) attracted its usual packed house of foreign and domestic visitors anxious to view the latest in fabrication equipment. And, as usual, attendees were not disappointed.

Perhaps the most exciting among the new products shown by Peddinghaus is a new heavy-duty bandsaw (model 38-18) specifically designed for the structural steel fabricator and capable of cutting shapes up to W14x730. The bandsaw is considerably faster than a cold saw and wastes less material.

Also new this year is an inexpensive Anglemaster (mdoel AFCPS 623K) for processing angles, flats and punching channel sections. It features an improved roller feed drive and measuring system to increase productivity. Another Anglemaster introduction is the "Ring of Fire" (model AFCPB-623), which features two punches and a three-axis plasma torch system to process angle, channel and flat stock. For more information on Peddinghaus products, please circle **92** on the reader service card in the back of this magazine.

In addition to the Peddinghaus products on display, more than a dozen other tool and software vendors displayed their wares.

Representatives from Design Data demonstrated their SDS/2 Steel Fabrication System, a modular system including: engineering, analysis & design; detailing; estimating; production control; and CNC. In addition, Design Data offered information on DesignLink, their electronic data interchange software product to transfer data from the design engineer to the steel detailer to the fabricator. For information on either SDS/2 or DesignLink, circle **32**.

CadVantage also was at the Oktoberfest and was demonstrating their PenWare system, which was first introduced earlier this year at the National Steel Construction Conference. PenVantage allows fabricators and detailers to enter information into CadVantage's detailing program with a pen, rather than using a computer keyboard or mouse. They also offered special discount packages, which they agreed to extend to MSC readers in November. For more information, circle **45**.

Dogwood Technologies demonstrated their detailing system, which includes modules for fabrication control, miscellaneous iron, material management, and billing. Circle **98**.

Steel Solutions showed their production systems for estimating, accounting, fabrication (mill orders, inventory, drawing control & bills of material), and for service centers, as well as provided a free demonstration disk. For information, circle **92**.

Acecad demonstrated its Strucad software, which has an entensive track record overseas. The new PC version of the program performs detailing and CNC operations for structural steel detailers. For information, circle **93**.

Another detailing package demonstrated at the Oktoberfest was CompuSTEEL. Its developers also were offering a special purchase plan. For information, circle **94**.

More specialized is MTC, which demontrated its CNC Profile Cutting Software. For information, circle, **95**.

Topping off the software vendors was SteelCAD, which demonstrated its detailing package. For information, circle **84**.

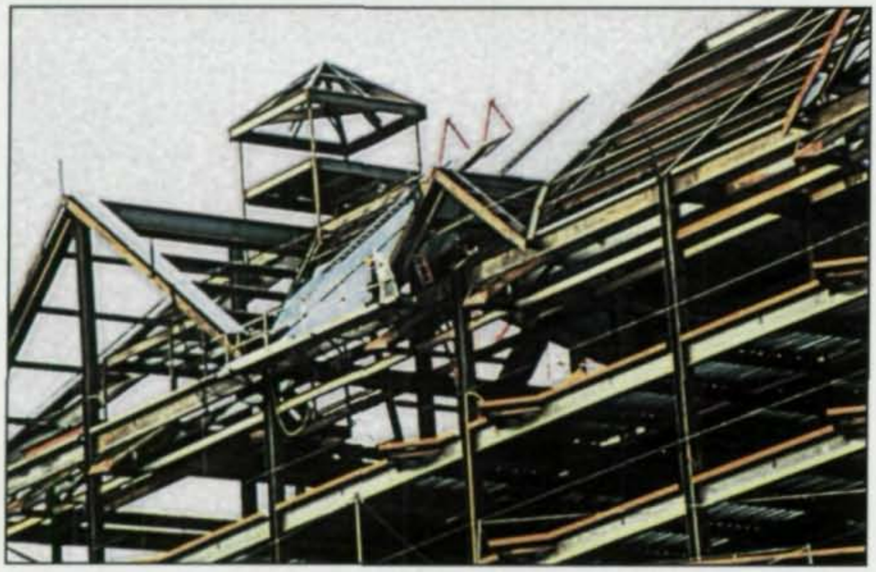
Other exhibitors at the show included: Pullmax (manufacturers of forming, bending, shearing, punching, rolling and beveling equipment for plates)—circle **96**.LeJeune Bolt Company—circle **73**; and J&M Turner (manufacturers of direct tension indicators)—circle **97**.

## T.R. HIGGINS LECTURES

Upcoming T.R. Higgins Lectures, featuring Lawrence Griffis, P.E., speaking on composite construction, are scheduled for Atlanta (Nov. 15), Bethlehem, PA (Dec. 8), St. Louis (Feb. 9), and Milwaukee (Feb. 10). Contact Robert Lorenz at 312/670-2400 or your local AISC Marketing Regional Engineer for more information.

# GAMBLING ON SUCCESS

## Due to the rampant popularity of casino gambling, speed of erection was a critical factor in the design of a massive addition to the Foxwoods Casino



*Because speed was crucial on this project, a structural steel frame was chosen for expansion project, which encompassed a five-level parking garage, casino, 300-room hotel, a theater complex, restaurants and shopping. Top photo by Russell Studios, Inc.*

**By Peter G. Celella, P.E.**

THE HOTTEST SEGMENT OF THE ENTERTAINMENT INDUSTRY TODAY is undoubtedly casino gambling. Where once only Nevada could boast legal activity, casinos can now be found in every region of the U.S. While much of this boom is occurring on "riverboats", many states are allowing land-based casinos to be built on Indian reservations. One of the most successful is the Foxwoods Casino in Ledyard, CT.

Even as the original 250,000-sq.-ft. facility was being opened by the Mashantucket Pequot Indian Tribe in mid-February 1992, it was obvious that demand would outstrip the small facility.

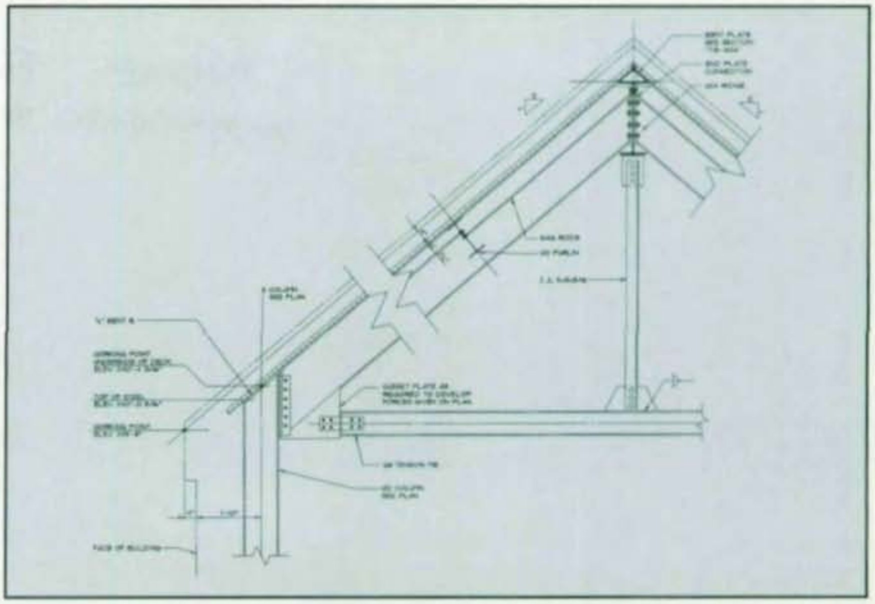
Phase IV, which opened in 1993, and Phase V, which opened this year, added a total of 1.5 million sq. ft. to the popular complex. Phase IV included a five-level, 1,600-car parking garage, and above it, a concourse/lobby area, a 50,000-sq.-ft. casino, and a six-story, 300-room hotel. The project also included areas for a theater complex, shops, restaurants, and other entertainment facilities. Phase V included a 20,000-sq.-ft. bus terminal, a 62,000-sq.-ft. food court, a 44,000-sq.-ft. office building for casino staff, and a 60,000-sq.-ft. multi-purpose room for daytime bingo and evening entertainment. A connecting bridge between the theaters and the food court also was constructed over a roadway that had to remain unobstructed during the construction period.

Adding to the complexity of designing a complex for such diverse uses, the lateral load-resisting systems for both Phases IV and V needed to be designed to resist both wind and seismic forces.

New England Design of Mansfield, CT, was the planning and design consultant on the project. Jeter, Cook and Jepson of Hartford, CT, was the architect and BVH Engineers, Inc., of Bloomfield, CT, was the structural engineer. Steel detailing, fabrication and erection was performed by AISC-member Berlin Steel Construction Co.

Virtually from the day it opened, Foxwoods Casino has operated near maximum capacity, 24-hours-a-day, seven-days-a-week. When BVH Engineers, Inc., began the structural design of the expansion, the original casino was already under operation, and every day spent on design or construction would result in a loss of projected revenues from an expanded facility. Obviously, an extremely accelerated construction schedule was the driving impetus for the project. Adding to the desire to increase revenues was a concern for minimizing the time period of the project's construction loan.

Because the project was fast-tracked—or "flash-tracked as it was often referred to during construction—it also was important that the choice of material for the structural framing have the flexibility to accomodate the inevitable design changes that were to occur before, during and after construction.



### Phase IV

The design for phase IV was complicated by the need to accommodate a wide variety of uses. The various parts of the structure, stacked one upon another and equivalent in height of a 15-story building, were being designed while the levels below were under construction. The design of Phase IV was started in May of 1992—at the same time that excavation began—and opened to the public on Labor Day Weekend of 1993, a mere 14 months later. The project required the use of 9,000 tons of structural steel, 700 precast double tees, and 52,000 cubic yards of cast-in-place concrete.

### Parking Garage

Because of a large grade differential, a 70-ft.-high concrete retaining wall was required along one side of the garage while the other side was left open. Because of schedule concerns, a common cast-in-place concrete garage was clearly not a practical alternative. Instead, the design used 60-ft.-long precast concrete tees supported on steel girders and columns. While the 60-ft. spans presented some difficulties for the framing of the non-parking levels above, that column spacing was chosen as the most appropriate for parking layout and circulation.

Steel columns were W14 sections, as large as W14x426, that were encased in concrete after the erection was completed. The girders supporting the precast tees were W24 sections, one on each side of the columns, and outriggered off the column centerlines in order to pick up tees framing in from each side. Clips with thru-bolts mechanically fastened the girders to the tees. After erection, the steel girders were sprayed with a cementi-

tious fire-proofing.

Lateral loads were transferred through the garage using steel braced frames that were later encased in concrete walls. At several locations, it was necessary to transfer uplift loads from the braced frame columns into the footings using high-strength anchor bolts with steel anchor plates.

While the costs were comparable with a conventionally constructed cast-in-place garage, the choice of a hybrid steel/precast system had several advantages. The primary advantage was the speed of erection. Precast tees were being cast while the design for the rest of the structure was proceeding and the foundations were being built. Once erection began, it took approximately two months to erect the entire garage. A secondary advantage of using this system was improved life cycle costs. New England winters—as in much of the northeastern and midwestern U.S.—are particularly hard on cast-in-place concrete parking structures because of an almost constant exposure to freezing and salts. Precast tees, due to pretensioning, are very resistant to cracking, and therefore road salts do not have a pathway to travel along to corrode the reinforcing. The cementitious fireproofing on the girders provides a degree of protection against corrosion, but in any case, since the girders are under the precast tees, they are not directly exposed to a corrosive environment.

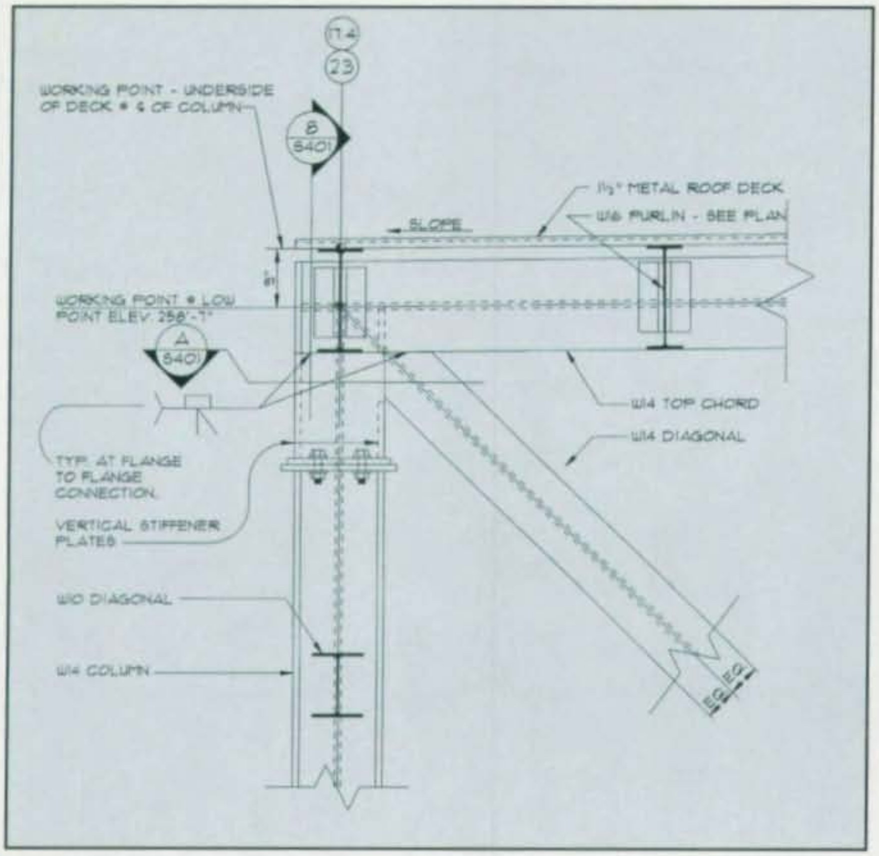
When scheduling was considered along with the life cycle benefits, the use of a hybrid system using a combination of precast tees and a supporting steel frame was determined to be the most economical long-term solution.

### Casino & Entertainment Facilities

Levels for the casino, theaters and shopping, along with the hotel lobby and operation support levels, were constructed immediately above the garage. The typical live load in the casino areas was required to be 250 psf due to the loadings imposed by the type of occupancy and gaming equipment. In order to electrify the floors for providing power to the slot machines and other gaming equipment, a double slab system (composed of a $5^1/_2$-in.-thick composite slab and a $4^1/_2$-in. electrified, light-weight topping) was chosen. In addition, an area used for hard count (coinage) storage required a design live loading of 700 psf.

Normally, spans of 30 ft. to 40 ft. would be used in areas with such high loadings. But since the column grid of 60 ft. was chosen for the garage below, and because of a need to maintain relatively long clear spans in the public use spaces, the same 60-ft. grid was continued through the casino and lobby levels. Conventional steel framing was used to frame these areas, but because of the 60-ft. spans and heavy loadings, W30 and W36 beams, 30-ft. on-center, were used.

Also constructed on these levels was an entertainment complex. It includes a 360-degree projection theater, a turbo-ride theater and a 1,300-seat Las Vegas type showroom. All of these areas were constructed of steel framing above the garage levels—i.e., the floors of the theaters were not slab on grade construction. Shop-welded steel trusses, 150-ft. long and 14-ft. deep, were used to clear span the required distance across the performance theater. The top and bottom chords on the trusses were W14 sections rotated horizontally. The trusses were shop fabricated with a construction joint in the span that allowed for field erection of only two pieces per truss. All of the shop fabrication was welded, while the construction joint was made using high-strength bolts. A penthouse, triangular in cross-section, was built on top of the theater roof to allow for the raising and lowering of a 30 ft. x 50 ft.




movie screen. When lowered in the middle of the showroom, the screen converts the space into a 300-seat Iwerks movie theater.

### Hotel

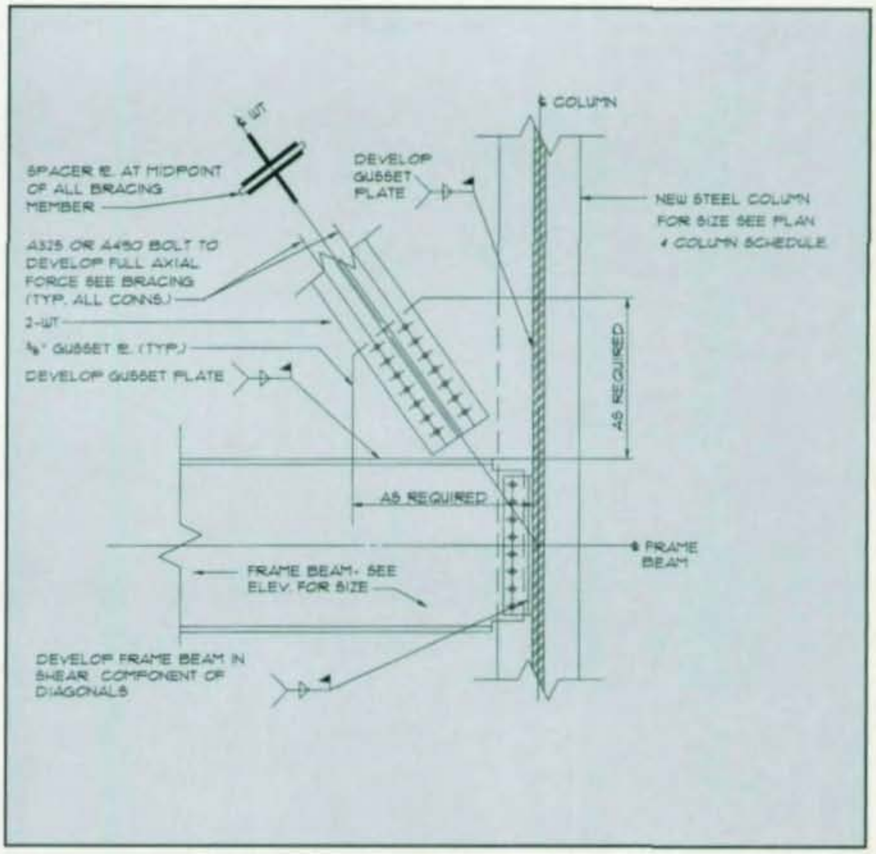
The six-story, 300-room hotel was erected on top of the casino and entertainment level. Cast-in-place or precast concrete is commonly chosen to frame this type of residential construction; but, given the aforementioned speed concerns, coupled with the difficulty inherent in building a concrete structure on top of a steel structure, it was decided to frame the hotel in steel.

The hotel, due to its footprint geometry, could no longer coincide with the 60-ft. column gird below. It was necessary to build a column transfer for the smaller spacings in the hotel. Although various alternatives were considered, including a staggered truss system, the column transfer was most easily accomplished by incorporating a series of transfer trusses into the bottom level of the hotel. Since steel lateral bracing frames were already being located with the demising walls of the hotel, it was simple to design these frames to include the extra gravity loading components necessary to span the 60-ft. column spacings below. These braced frames were used in combination with moment frames in the perpendicular direction to provide the lateral support for the hotel portion of the project.

Typical columns in the hotel were W14 sections. The moment frame beams were W21 and W24 sections, while the floor beams were composed of light W12 and W16 sections with W21 girders. The roof of the hotel was gabled and framed with W24 sloping girders and W12 beams with sag rods.

### PHASE V

Although the design of Phase




*Top photo by Robert Benson Photography*

V was not fast-tracked in the same manner as Phase IV, speed of construction was still extremely important. As part of this phase, the existing bingo hall was converted into a slot machine hall simultaneous with the construction of a new bingo/multipurpose room. It was important that this work occur in a timely manner, with a minimum of disruption to the existing facilities. Design began in September of 1993, erection started in November of 1993, and the doors opened for business in May 1994—a time span of only nine months from first pencil on paper to the opening of the new facility. A total of 2,000 tons of structural steel were utilized in Phase V.

### Bingo/Multi-Purpose Room

The bingo/multi-purpose room was spanned with 14-ft.-deep, 160-ft.-long steel trusses. These trusses were fabricated and erected in a manner similar to those in the Phase IV theater. The bingo hall trusses also included a catwalk system servicing the lighting and sound systems used at boxing and musical events. The framing for the catwalks had to be coordinated and incorporated into the bottom chord bracing provided for the trusses. Mechanical mezzanines were located to either side of the room, at the level of the bottom chord, for the placement of fans for ventilating the large open space.

### Bus Terminal/Food Court

The food court floor had columns located on a 28-ft. x 28-ft. grid. Typical beams were W18x35 sections and the girders were W24x76 sections. The roof framing was designed to support 28-ft. x 56-ft. skylights. A canopy cantilevering 28 ft. out from the structure was provided to shelter patrons being dropped off at the bus terminal below. A tubular hangar runs from the outside edge of these canopies to the exterior building columns.

### Office Building

A three-story office building for the use of the casino administrative staff was erected on top of the food court. The floors were framed using W16 beams and W24 girders. Lateral loads were transferred using a series of braced frames. The roof framing included a series of sloped screening and curved roof penthouses.

### Conclusion

The massive additions to the Foxwoods Casino were a unique and challenging project. A very large quantity of structural steel was raised in a very short period of time. The structures boasted a number of unique uses and configurations that offered many opportunities for investigating creative solutions to engineering problems. The most important reason for being able to accomplish such a feat of rapid design and erection is the extremely close cooperation and coordination of all the project's consultants.

*Peter G. Celella, P.E., is a project manager with BVH Engineers, Inc., Bloomfield, CT.*

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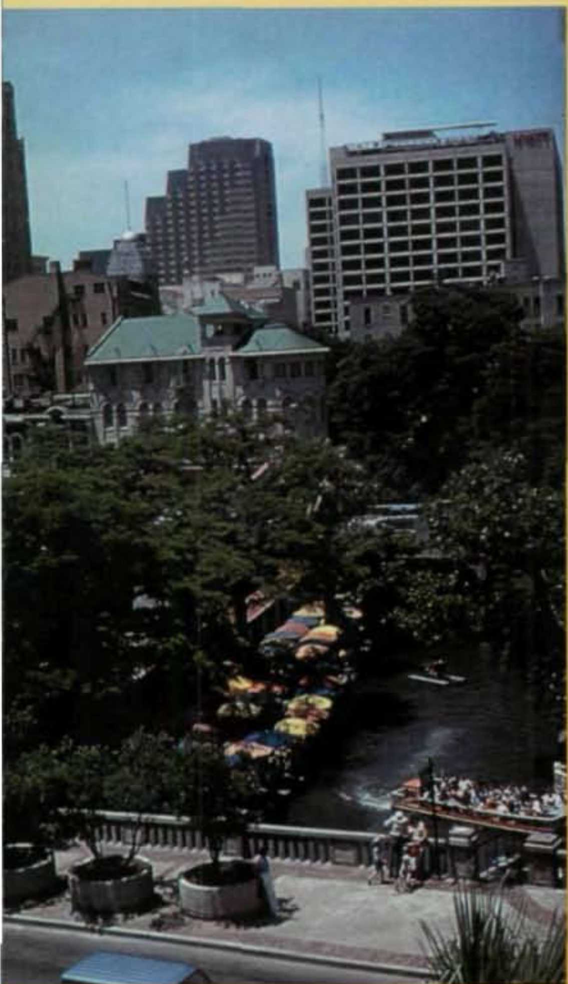

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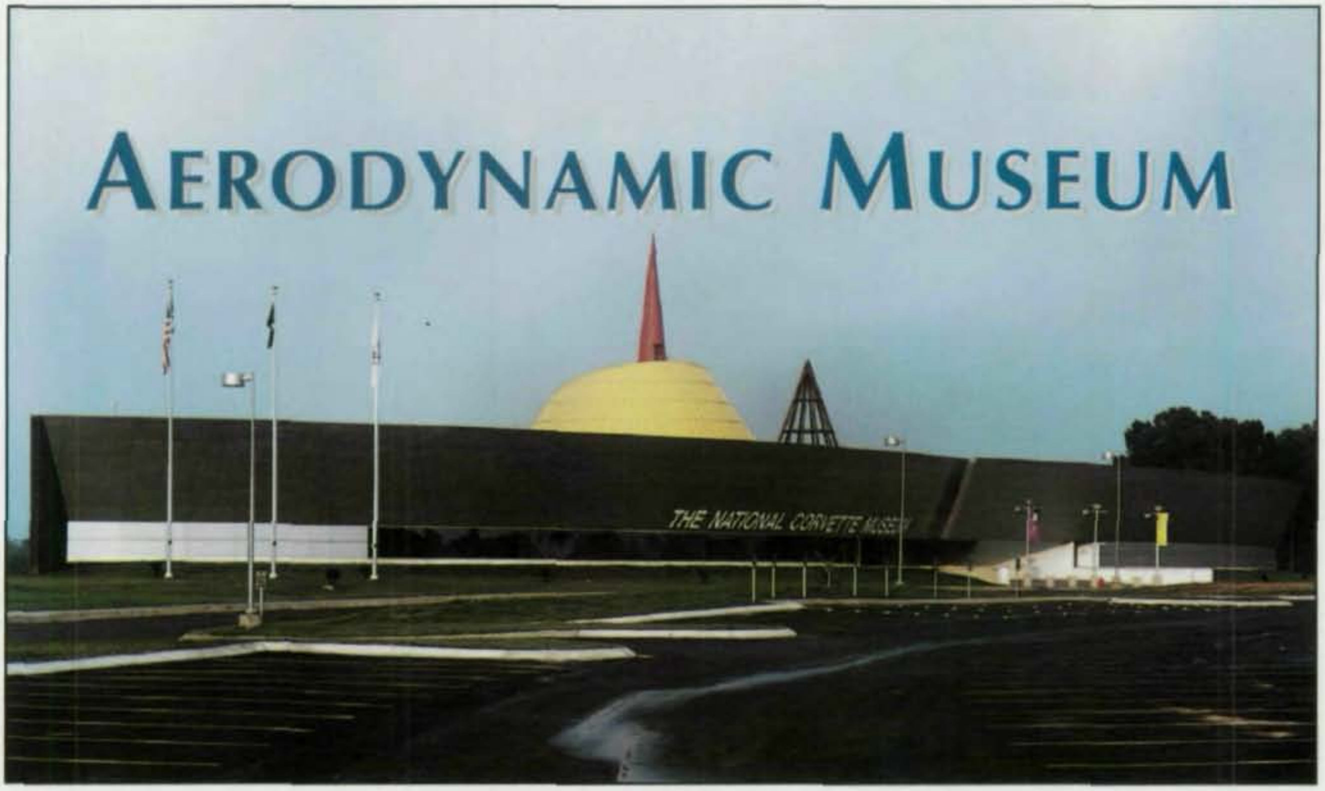
# AERODYNAMIC MUSEUM



## The design of the new National Corvette Museum is fittingly as sleek and stylish as is its subject

**By Mehdi Setareh, P.E., and William Lefkofsky, P.E.**

**Photography by Timothy Hursley**

WITH ITS SLEEK, CURVING FACADE, THE NEW NATIONAL CORVETTE MUSEUM in Bowling Green, KY, evokes an appropriately aerodynamic image. The museum houses an exhibit on the 40 year history of the Corvette car—the first time a museum was devoted entirely to a single car model.

Funded by donations from Corvette enthusiast groups and General Motors Corporation, it includes a theater, automobile display area and a pavillion for car display. The budget for the entire 65,000-sq.-ft. complex was only $8 million, which makes its construction costs very economical compared with other museums. The museum is expected to attract approximately half a million visitors annually to its displays of more than 50 Corvettes, including several limited edition models. In addition, the museum will house a special driving simulator.

The building's striking geometry is the result of a design by the architectural firm of Neumann/Smith and Associates, Southfield, MI. "This is not a typical quiet museum, but a theatrical one, in which the building itself is part of the show," explained Kenneth Neumann, the design architect. "Not a quiet place but a provocative one."

The entire complex includes three major zones interconnected by an atrium. The office atrium portion, which exhibits the history of the Corvette since its creation in 1953, is steel framed, using 8x8 tubes for the columns, W8x35 sections for the beams, and steel joists for the roof.

General contractor on the project was a joint venture of Alliance/Turner. Steel fabricator was AISC-member Grace & Wylie Fabricators, Inc. Erector was Al-Tenn.

### COMPLEX EXHIBIT SPACE

THE EXHIBIT AREA ALSO IS STEEL FRAMED, though the design is more complex. It features a saw-toothed roof profile reminiscent of original automobile manufacturing facilities. The structure includes exposed steel beams and stiffeners, which frame into exposed steel tube column stubs over concrete column bases. At the north end of the building, a 14-ft.-deep truss is tilted out-

board 60 degrees and spans 54 ft. with a 16-ft. cantilever. The truss is formed from both 8x8 tube and W8x48 members. The rest of the exhibit area is framed with 8x8 tubes and joists and joist girders.

On the exterior, the cladding is curved, which gives a circular appearance to the truss. The exhibit area will include about 25 different models of the car and provide information on the design and technology, including showing cut-aways of engines, mold shapes, scenes from racing tracks, gas station models, an assembly line and car crash testing models. The exhibit area even includes an area for purchasing a new car.

### COMPLEX DESIGN

THE FINAL AREA, THE CAR DISPLAY PAVILLION, is the most complex. It includes a 60-ft.-high frustum-shaped steel structure with a 15-degree monoslope roof.

This portion is used for the display of about 20 different models of Corvette cars. Also, from the inside of the pavillion, a 108-ft.-high spire extends out of the frustum. This has a bright yellow color, so that it can be distinguished from afar, and it's topped with a red light, which is suggestive of the tail-light of a Corvette. In the spire, pictures and photos of historic events during the last 40 years, including different car models, are displayed. Both the pavillion and the spire have natural light from skylights.

The structural design of the pavillion was the most difficult part of the project. Because the structure was not symmetric, and therefore, it was not possible to reduce input data. However, a computer finite element model, using beam elements for the vertical and horizontal members and plate elements to represent the siding panels, was created of the entire structure and STAAD-III was used for analysis and design.

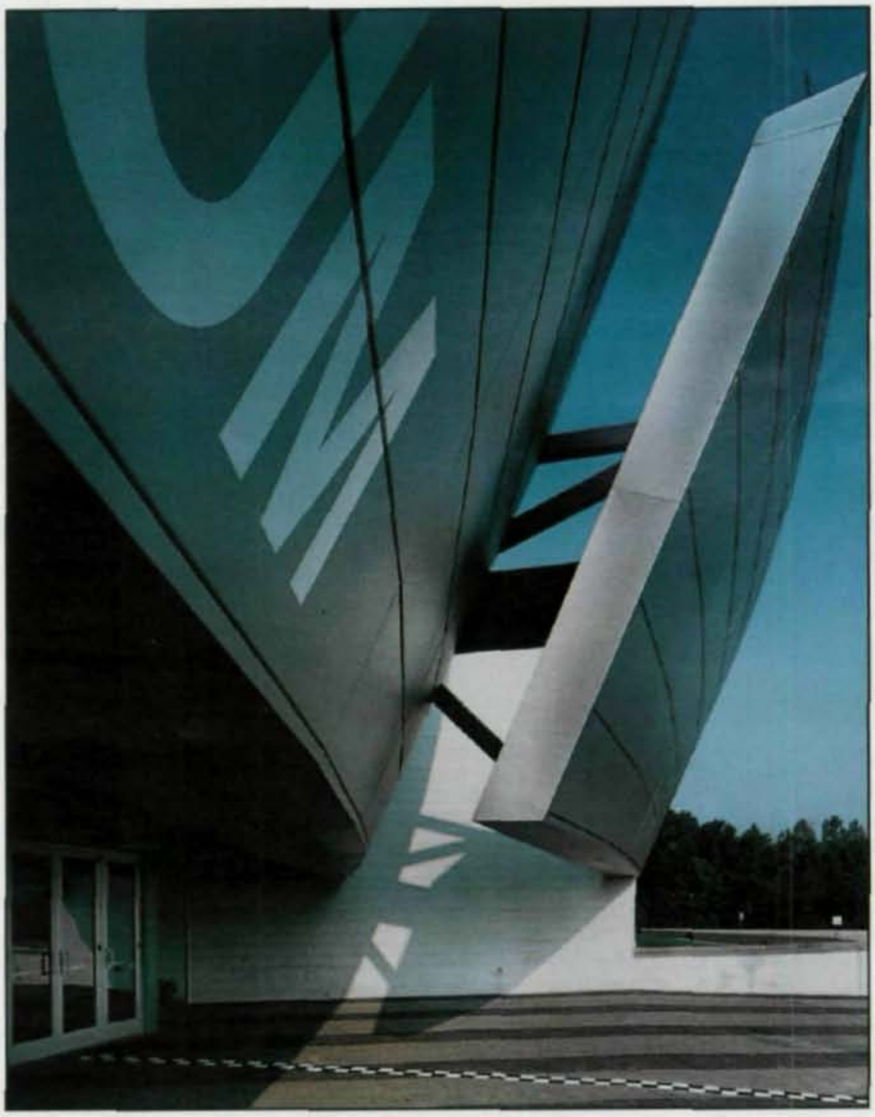
Wind was the main structural design factor in determining member sizes and lateral stiffness requirements. While the aerodynamic shape of the building will reduce wind drag, this factor was neglected to arrive at a conservative overall factor of safety for the structure. Also, to prevent damage to the skylight, the relative movements of points on the structure was limited.

### Mixing Tubes And Wide Flange

Due to the curved configuration and architectural requirements, the entire structure was originally designed using exposed 6-in. and 8-in. diameter structural pipes for the horizontal girts and 10-in. structural pipes for the vertical members. However, due to budget constraints resulting in part from the high cost of the ground-smooth welded connections of the vertical and horizontal members, this plan had to be modified.

Various schemes were considered to reduce costs, with the final design featuring W12x40 (A36) vertical members and 6-in. and 8-in. pipes for the horizontal girts. As a result, the connections of the horizontal and vertical members was greatly simplified. Plates were shop-welded to the horizontal girts, which in turn were field bolted to the vertical members.

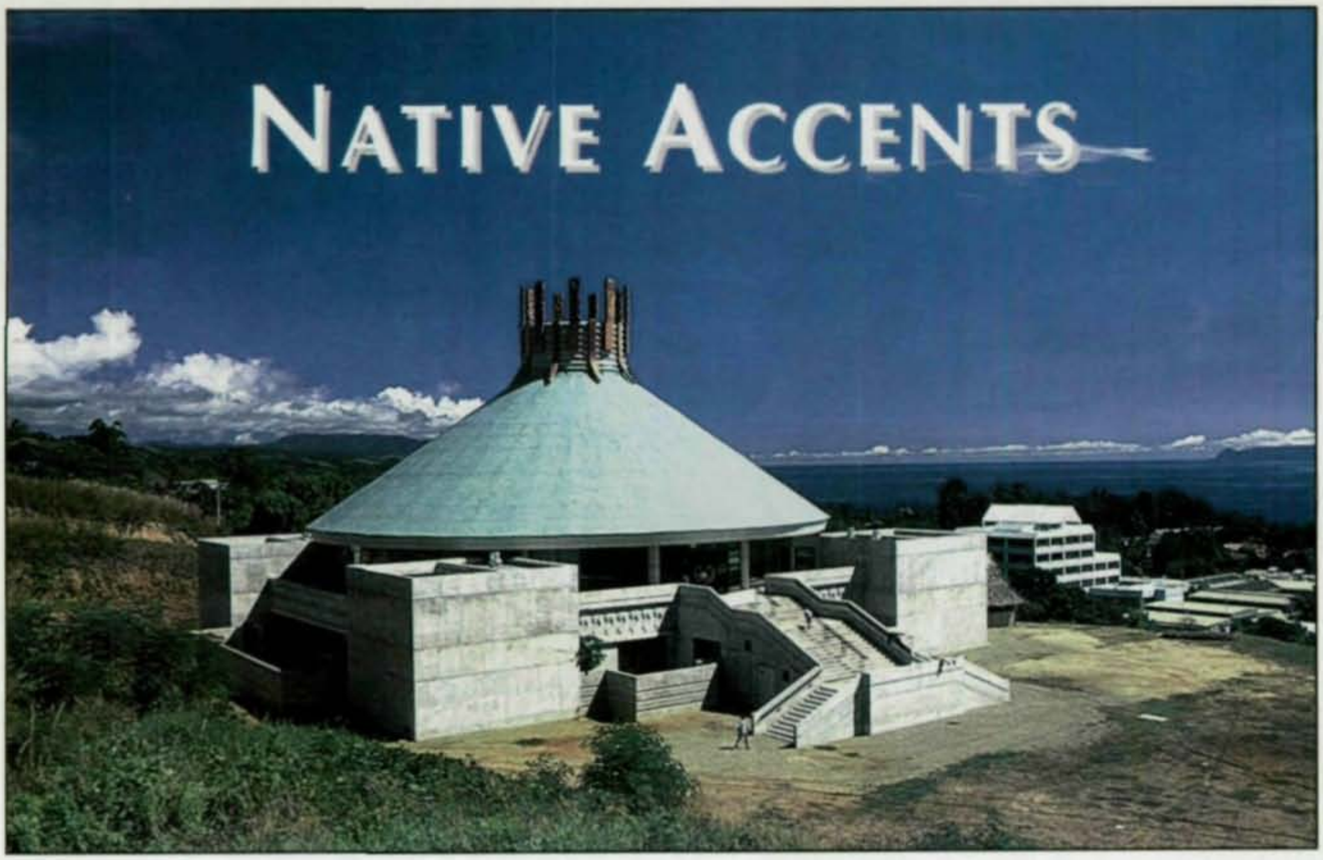
The fustrum includes a compression ring, formed from 12-in. pipe, at its base and a tension ring at top. The compression ring supports all of the sloped vertical and horizontal members. The tension ring supports two welded trusses, which support the structure's roof. The trusses, which are oriented perpendicular to each other, are made of 12-in.



pipe and are connected to the frustum by end plates welded to the pipes and field bolted to the vertical members of the structure, which consist of W8x28 sections. The total weight of the structural steel for the frustum alone was 93 tons. In the case of the spire, the lateral loading was not significant because most of the structure is enclosed by the frustum.

*Mehdi Setareh, P.E., Ph.D., is a consulting engineer and associate professor at the colleges of architecture and engineering, Lawrence Technological University, Southfield, MI. William Lefkofsky, P.E., is president of L&A Inc., a structural engineering firm located in Southfield.*

*The exterior of the National Corvette Museum features an obviously appropriate sleek, aerodynamic appearance, while the interior offers strong exhibition spaces. Photography by Timothy Hursley*



# NATIVE ACCENTS

## Structural steel proved to be the best material to reinterpret a traditional Solomon Island form into a modern Parliament Building

**By Steven M. Baldridge, C.E., P.E., S.E.**

ON AUGUST 7, 1942, THE U.S. MARINES LANDED IN THE SOLOMON ISLANDS at Guadalcanal, beginning what was to become a hard-fought six-month battle critical to the outcome of World War II. During the ensuing days the U.S. Navy would suffer one of its worst defeats. Under the cover of night the Japanese would position their fleet to open fire at point-blank range on the U.S. and Australian naval forces.

Off the coast of Guadalcanal around Cape Esperance and Savo Island this area would become appropriately named "Iron Bottom Sound." Tens of thousands of tons of steel would be laid to rest during this battle, whose eventual toll included more than 50 ships and planes belonging to the U.S., Australia and Japan. In the end the Allied forces would persevere and the Battle for Guadalcanal would become a pivotal victory in the Pacific War. Historians have said that the men who had fought there "bore an aura of endurance" which veterans of almost no other Pacific Campaign acquired.

It was in this historical context that Congressman Stephen Solarz (D, New York) sponsored an amendment to the 1990 Defense Appropriation Bill providing funding for a $5 million Parliament Building for the Solomon Islands. Congressman Solarz said the building will give "tangible support for democracy *in that part of the world*" and will be a fitting monument to the American G.I.'s who died in the Battle of Guadalcanal. The building would be built as a gift from the United States in commemoration of the 50th anniversary of the World War II Battle of Guadalcanal.

### Architectural Design

THE PROJECT WAS ADMINISTERED BY THE Department of the Navy, Pacific Division Naval Facilities Engineering Command, who selected the Honolulu, HI-based architectural firm Wimberly Allison Tong &

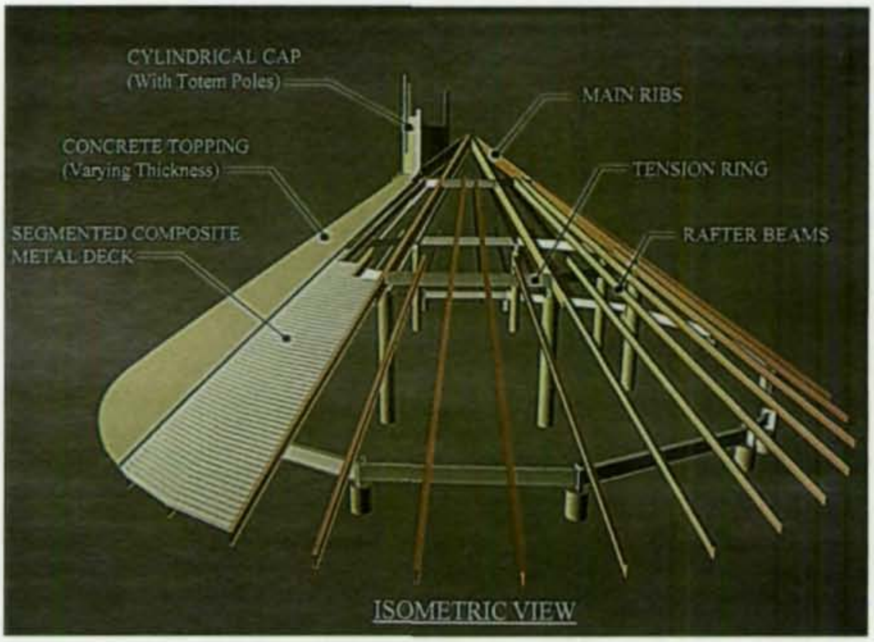
Goo to design the Parliament Building.

WAT&G was founded within a few miles of Pearl Harbor at the end of World War II and has been a pioneer since the 1950s in the design of environmentally sensitive projects in more than 50 countries. Their experience in the Pacific vernacular has included projects in Bora Bora, Pago Pago, and Fiji as well as in the Hawaiian Islands.

Project designer Michael J. Batchelor, RIBA, AIA, said that in describing what they wanted in the design of a Parliament Facility, Solomon Island officials requested that it be representative of their emerging democracy and that it should be "essentially Solomon Islands in style, not an imposed architecture."

The design solution is a two-story, 22,000 square-foot building of reinforced concrete and steel frame with extensive glazing. Its shell roof is an abstracted version of two local roof styles—those of Tamotu and Guadalcanal provinces.

The roof's defining conical shape is derived from native Tamotu roofs and has the unusual ridge characteristic of indigenous Guadalcanal roofs. Detail at the top is unique to the Solomon Islands. It has seven major elements symbolizing the seven provinces of those islands.




*The isometric view and a construction photo reveal the design of the conical-shaped roof of the new Parliament Building.*

### System Selection

FOR THE PROJECT'S HONOLULU-BASED STRUCTURAL ENGINEER, Martin & Bravo, Inc., the most challenging aspect of this project would be the design of the centerpiece Conical Shaped Roof. Not only did this roof have to meet the extreme structural load criteria of a region known for both severe typhoons and earthquakes, but the structural system selected had to take into account the remoteness of the project location as well.

The roof in its completed form would have an overall diameter of 37.1 meters (121.7 ft.) with a rise of 13 m (42.7 ft.) to its apex. The functional requirements of the Debate Chamber of the Parliament Building meant that the conical-shaped roof would have to be column free for the interior 18 m (59.0 ft.) diameter of the roof. Its geometry is further complicated by a perimeter cantilever extending a minimum of 2.8 m (9.2 ft.). While these dimensions would not be considered difficult by standard U.S. construction practice, in a location as remote as the Solomon Islands, more than a two-day series of flights from Honolulu, this geometry would complicate the goals of the constructibility and economy of the final design.

With the aid of cost consultants Rider Hunt Ltd., structural steel was chosen to frame the Conical Shaped Roof of the Parliament Building. Julian Anderson of Rider Hunt Ltd., summarized the selection as follows:

"Structural steel was chosen for the framing system because it was able to handle the spans required by the Architect and achieve the desired roof profile. Conventional poured-in-place concrete was not an option because it could not easily achieve the spans without excessively-sized members. Post-tensioned concrete was not an option because there was insufficient skill and quality control in the area to ensure that work could be carried out properly. On the other hand, the structural steel is cheaply imported from Korea, New Zealand and Australia, and can be erect-

*Shown above is the apex of the Parliament Building, while an interior view of the space is pictured on the opposite page. As with the exterior, the interior features traditional forms and designs.*

ed with imported labor over a short period of time and at a relatively modest price."

### Design Criteria

THE PROJECT WAS DESIGNED TO CONFORM WITH APPLICABLE Solomon Island Building Codes, which is a combination of the Australian and New Zealand Building Codes. All drawings were required to follow metric standards.

The design wind loads followed Australian Standard AS 1170.2, a detailed 96-page document devoted entirely to wind loads. With a Basic Wind Speed of 60 m/s (134.2 mph) the equivalent static pressure acting on the sloped surface of the roof was 2.50 kPa (52.2 psf) inward, 3.45 kPa (72.1 psf) outward. At the cantilevered portion of the roof the design loads were as high as 8.90 kPa (185.9 psf).

For seismic loads New

Zealand Standard NZS 4203 was used. The total horizontal seismic force for the structure was equivalent to 0.32 times the total reduced gravity load (approximately one-third greater than an equivalent structure in UBC Seismic Zone 4). Based on these seismic loads the structure was required to be designed to create a continuous load path capable of carrying 3260 kN (732.9 kips) of horizontal force from the roof down to the foundation.

In addition to the lateral forces, significant gravity loads had to be carried by the steel roof frame as well. The roof system is comprised of composite metal deck with concrete topping enveloped by a waterproofing system and an elaborate interior architectural ceiling which includes mechanical equipment, catwalks and extensive architectural ornamentation. The combination of loads from the structural and architectural components is a heavy 4.10 kPa (85.6 psf). Near the apex of the roof the additional weight from a cylindrical cap carrying nine wood "totem" carvings, each one weighing as much as 9 kN (2 kips) had to be accommodated by the steel frame as well.

**Structural System**

THE STRUCTURAL SOLUTION TO THE CHALLENGING GEOMETRY, LOADS AND PROJECT LOCATION was to utilize the interior ring of eight uniformly spaced columns to support a Steel Ribbed Dome. The Ribbed Dome consists of eight identical straight wide-flange 250 MPa yield (equivalent to Grade 36) 610 UB 101 (W24 x 68) "ribs" interconnected at a compression connection at the apex of the cone and stiffened by a 530 UB 82 (W21 x 57) tension ring at the interior circle of columns. The remainder of the roof is framed with 360 UB 45 and 360 UB 57 (W14 x 30 and W14 x 38) rafter beams spanning from the interior tension ring to the perimeter beam where they cantilever out to the outermost edge of the roof.

The cone shape is first approximated by segmenting the steel framing in plan and elevating each connection of the rafter beams above the segmented ring of support beams to match the cone profile. The cone is then completed by varying the thickness of the concrete topping on the segmented composite metal deck. Segmenting of the steel provided a cost savings by eliminating the need to bend the structural steel.

In order to keep the cone shape "true", special considerations had to be made to accommodate differences in the relative deflections of the various roof framing members. For example, deflection control was complicated at the perimeter edge of the roof by a combination of varying rafter beam cantilever lengths and support beam deflections. Inconsistencies in the final edge deflection were balanced by varying the rafter beam member sizes (stiffness) and cambering the support beams.

Lateral loads follow a load path which includes a horizontal channel attached to the top of the perimeter beams to provide the weak axis strength required to transfer loads coming down from the rafter beams out to a steel stub column tied to the structure below. Since the exterior columns were much shorter and therefore much stiffer than the interior columns, the majority of the roof's lateral loads were designed to transfer out at this location.

*Another view of the Parliament Building's interior.*

### Construction

IRONICALLY, THE CONSTRUCTION OF THIS PROJECT included representatives of each of the countries which had fought on Guadalcanal 50 years earlier. The general contractor on the project was the Japan-based Kitano Construction. The United States was represented by the design team and Australia by the subcontractor for the fabrication and erection of the structural steel.

The teamwork required to successfully complete this project is an example that as time passes and our world grows smaller, historical and cultural differences diminish as well. The construction was completed within budget and the Solomon Islands Parliament Building was turned over to the people of the Solomon Islands in a ceremony which included U.S. dignitaries and veterans in November of 1993.

*Steven M. Baldridge, C.E., P.E., S.E. is a structural engineer with Martin & Bravo, Inc. Consulting Structural Engineers in Honolulu, HI.*

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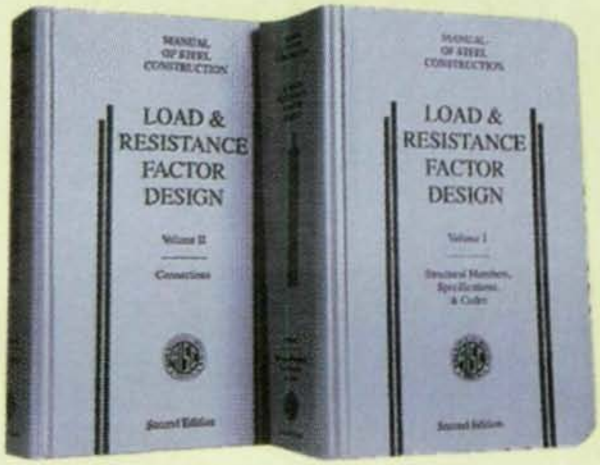


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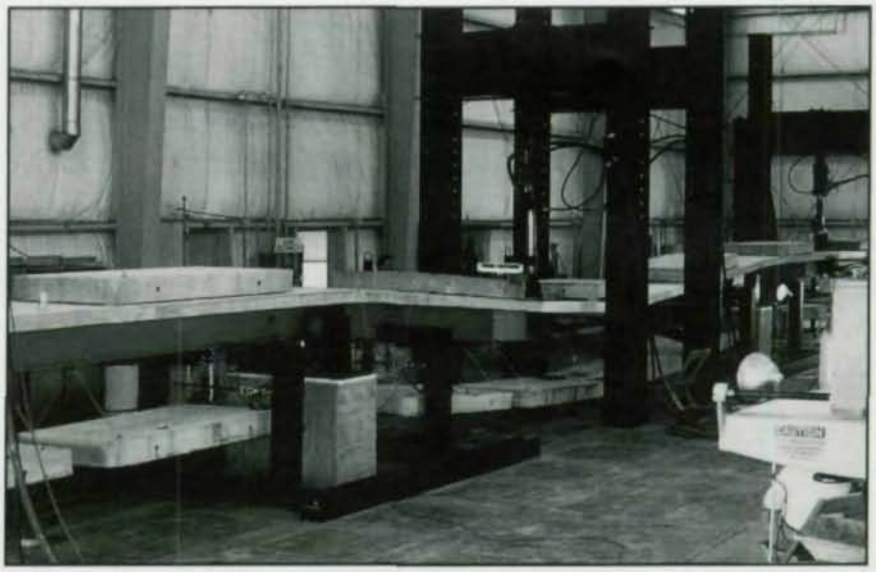
# Large-Scale Girder Testing For More Economical Steel Bridges

by Michael G. Barker, P.E., and Bryan A. Hartnagel

RECENTLY, A 98-FT. LONG, THREE-SPAN (30ft.-38ft.-30ft.) composite steel bridge girder was tested in the Remote Test Facility at the University of Missouri-Columbia. The experimental test is part of a three-year project to develop advanced inelastic design provisions for steel girder bridges.

The study's primary objectives are to verify design limit behavior of current inelastic design provisions for compact shapes and to extend the inelastic procedures to include noncompact plate girder designs. Inelastic design procedures allow the designer more flexibility and the possibility of more economical designs by eliminating cover plates and flange transitions over negative moment regions.

The study is part of the joint venture Innovations in Steel Construction sponsored by the National Science Foundation, the American Iron and Steel Institute, and the American Institute of Steel Construction. The Missouri Highway and Transportation Department is also supplying funds for the test girders. In addition, major improvements in laboratory facilities were possible through industrial support from Bethlehem Steel, US Steel, Nucor-Yamato Steel, St. Louis Screw & Bolt Co., and AISC-member Stupp Bros. Inc., St. Louis. AISC-member Delongs Inc., Jefferson City, MO, fabricated the steel beam.



*Overall view of beam during the plastic collapse test. Compensatory dead load concrete blocks were hung from the beam to simulate modeled dead loads. Concrete blocks on top represent composite dead loads.*

In total, three approximately 100 ft long girders will be tested: one representing a rolled shape compact girder (the above test) and two representing plate girder designs with thin webs (noncompact). This article discusses the results of the first girder test.

### Inelastic Bridge Design Procedures

Alternate Load Factor Design (ALFD) procedures (inelastic design) were adopted by AASHTO in 1986. The procedures account for the reserve strength inherent in multiple-span steel girder bridges by allowing redistribution of interior pier region elastic moments to adjacent positive moment regions. The design procedures specify requirements at service load levels (normal traffic), overload levels (occasional heavy vehicle), and maximum load levels (one-time maximum vehicle).

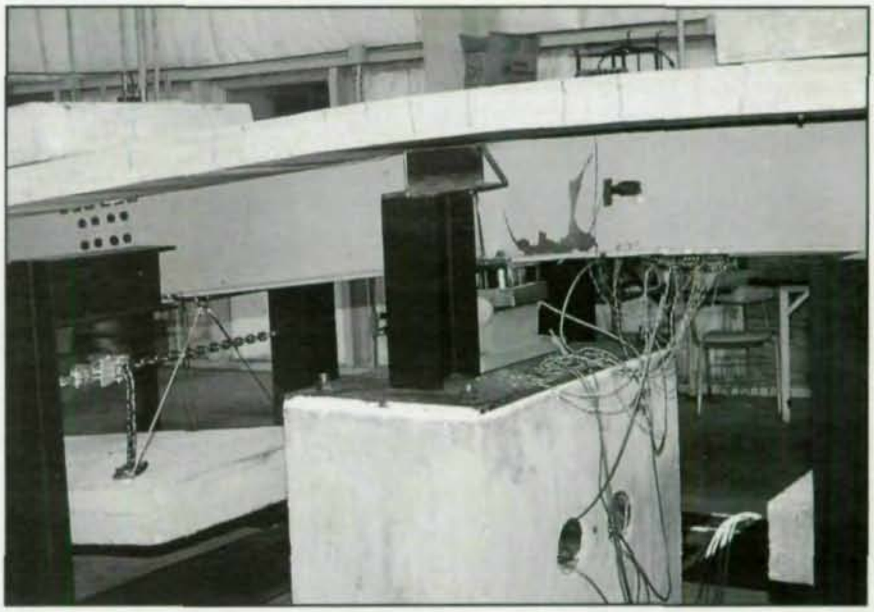
ALFD procedures can result in more economical designs. However, current ALFD provisions apply only to steel beam bridges with compact sections. For more economical and more consistent designs for all types of steel bridges, the ALFD provisions need to be extended to include noncompact sections.

### Test Girder

The test girder was a one-half-scale model of an interior girder from a three-span (60ft-76ft-60ft) four-girder composite bridge. The design was based on inelastic design provisions from the Fourth Draft of the proposed Load and Resistance Factor Design (LRFD) Bridge Design Specifications, which incorporate the ALFD provisions. The test girder consisted of a W14x26 continuous steel shape with a fully-composite 50.5 in wide by 4 in thick concrete deck. The steel material was A572 Grade 50 and the concrete design strength was 4000 psi.

A one-half-scale model only weighs one-quarter as much as the prototype. Therefore, compensatory concrete blocks were hung from the bottom of the W-shape before placing the concrete deck to restore the self-weight lost due to the scaling. Additional concrete blocks were placed on top of the deck after it hardened to represent the composite dead loads (wearing surface, barriers, etc.).

Modeled truck loads were applied by four individually controlled hydraulic rams. One ram was in each of the two outer spans and the other two were located in the center span. The rams were synchronized to simulate a truck traveling across the structure. The truck load sequence could be linearly adjusted to represent any percentage of the modeled truck design weight (LL).



*Close up of north interior pier showing large inelastic rotation. Also shown are the lateral bracing, reaction load cell and pin support, and strain gage instrumentation.*

### TEST PROGRAM

Load sequences were applied to the test beam cyclically at various load levels. Experimental measurements were recorded throughout the testing. The service load, overload and maximum load design levels were rigorously examined to verify the girder behavior at the design limits. Afterwards, the girder was tested to failure (center span) to examine the load-deflection response and to determine the collapse capacity.

Elastic low-level tests were carried out at 10, 20, 40, 60, 70, 80, and 90%LL. Lower load levels provided the opportunity to confirm elastic behavior and instrumentation performance. Service level loads (100%LL) were applied to validate fatigue and deflection requirements of the LRFD provisions. Increasing the loads towards the overload level, loads of 110 and 120%LL were applied to chart the behavior in this modeled truck weight range.

At the overload level (130%LL), the girder experienced significant inelastic behavior. This is characterized by residual deflection or permanent set. Design provisions predict this residual deflection and limits stresses in positive moment portions of the structure.

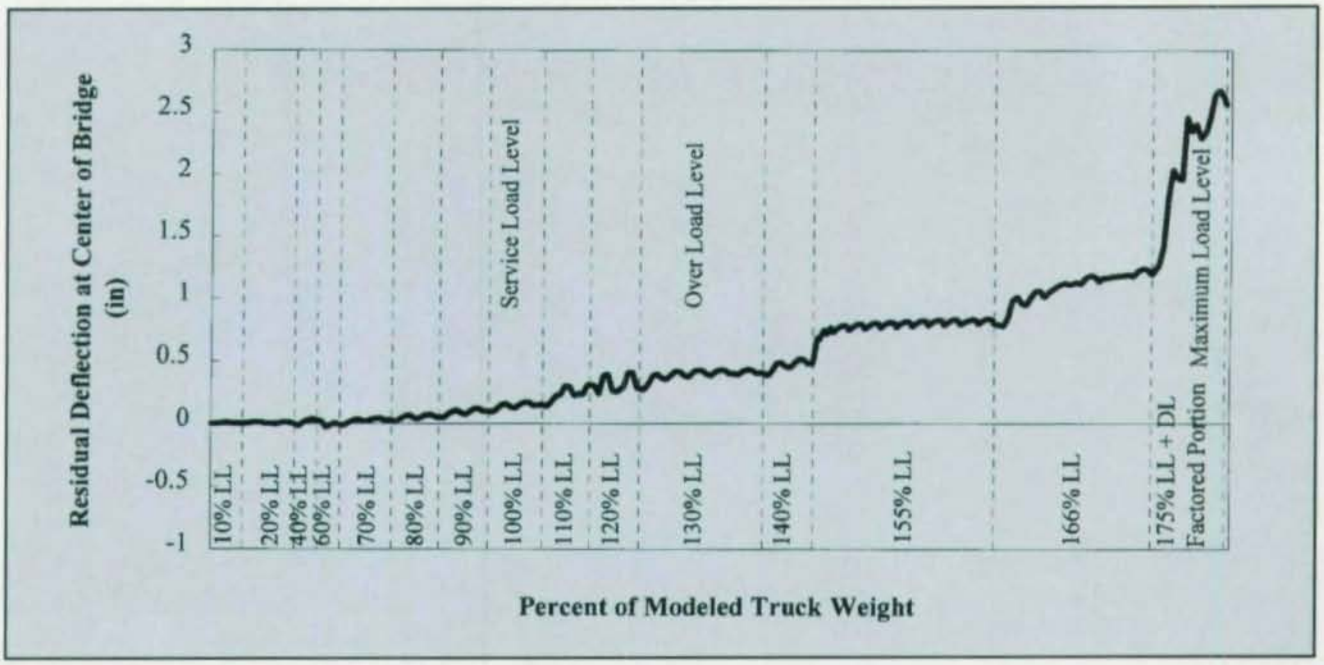
Cyclic loads were applied at 140, 155, and 166%LL to examine the inelastic behavior above the overload level. The last simulated moving truck load was at the maximum load level (175%LL plus additional dead load). This loading represents the worst possible maximum design load level applied to a bridge. After the cyclic tests, the girder was tested to failure by monotonically increasing loads proportioned to represent the theoretical design collapse configuration.

### TEST RESULTS

The girder performed well throughout the entire test. The elastic and inelastic response of the test beam met all the LRFD inelastic design requirements. The following compares the model response to the predicted prototype design response and presents results of this test.

Permanent set was first observed at 70%LL. This corresponds to an expected first yield at 67% when considering residual stresses in the prototype girder. As the modeled truck weight increased, the residual deflection grew. After the last cycle of the maximum load level (175%LL plus additional dead load), the total permanent set was 2.56 in.

At service level loads (100%LL), the experimental live load deflection was within 5% of the theoretical modeled deflection. Fatigue requirements were




*The permanent set during the moving load tests grew as the modeled truck weight increased. The residual deflection stabilized at all loads except at the maximum load level.*

At the overload level (130%LL), the measured maximum positive moment stress (derived from strain) satisfied the design requirements and matched the theoretical stress within 14%. The residual deflection was 0.38 in. which was significantly larger than the LRFD inelastic design provisions esti-

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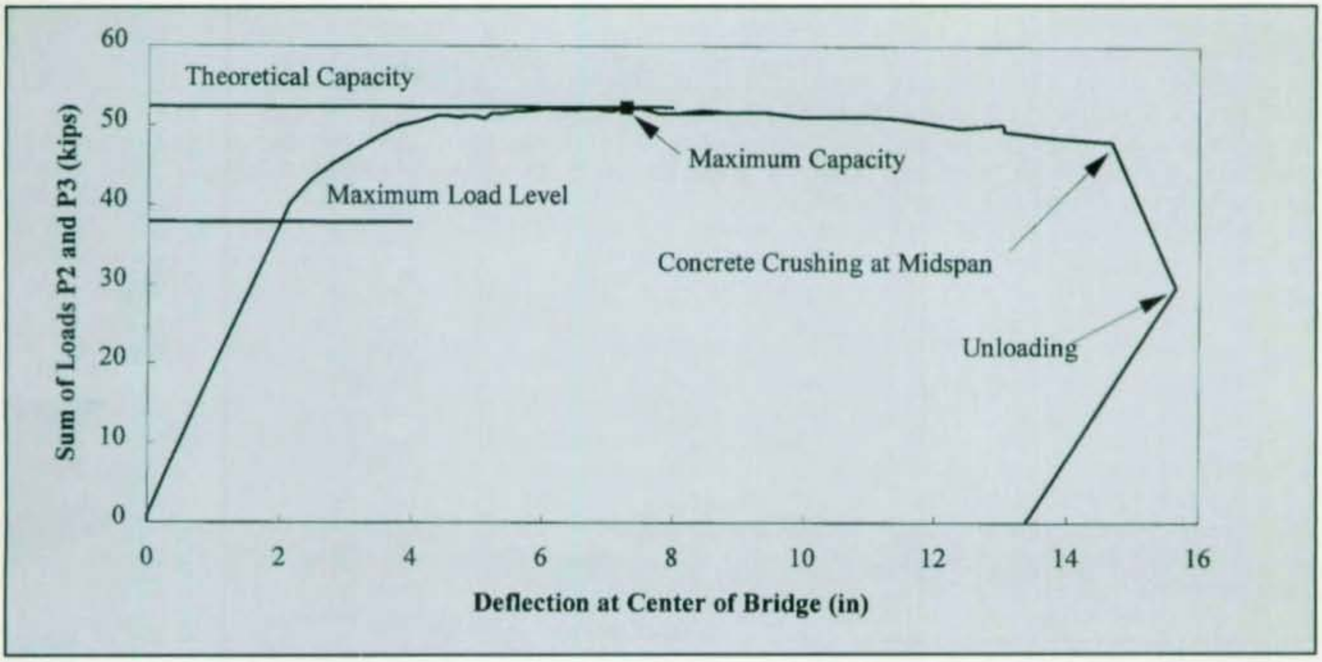


mated. However, using recent improved moment-rotation behavior models at the piers, the residual deflection was estimated within 5%.

The experimental plastic collapse capacity exceeded the maximum load level (175%LL plus additional dead load) by 34% consistent with the design. The

*The plastic collapse test showed tremendous ductility at near maximum loads. The ultimate capacity was well above the maximum load level as designed.*

experimental model collapse load and the theoretical collapse load matched within 1%. The girder maintained nearly ultimate loads up to 14 in. of deflection (in addition to the 2.56 in. during the cyclic tests). This deflection (length/deflection =33) shows tremendous ductility for this compact girder.

### SUMMARY

The test results reported herein give an overview of the general elastic, inelastic, and plastic behavior of the one-half-scale three-span composite test beam. The experimental results compared well with theoretical expectations. However, the LRFD inelastic design provisions significantly under-estimated the overload level (130%LL) residual deflections. For inelastic design, this permanent set would be cambered out along with the dead load deflections. Therefore, it is a fairly important quantity. Using more current behavior models, the residual deflection was accurately estimated. Future analysis of this test and others will yield insight into the best approach for estimating these deflections.

The plastic collapse test illustrated the ductility in compact composite beams. The measured collapse was within 1% of the predicted ultimate capacity. The primary reason that the beam behaved so well is that it is compact with the flanges being well below the compactness requirements (ultra-compact). This will not be the case for the second two girder tests. The flanges will still be ultra-compact, but the web will have typical plate girder width/thickness ratios. However, previous work has shown that, although these girders are not as ductile, the noncompact sections have predictable moment-rotation behavior that can be incorporated into inelastic design provisions.

More economical steel bridge designs can be realized using inelastic design provisions. The results of this test and others validate these procedures for bridges with compact girders. Provisions for the inelastic design for bridges comprising noncompact sections would be very beneficial. However, even though the analytical tools exist for inelastic design of these girders, large-scale testing is necessary to validate theoretical engineering practice. The second two noncompact girder tests from this project will provide vital information for the development of these provisions.

*Michael G. Barker, P.E., is an Assistant Professor and Bryan A. Hartnagel is a PhD student at the University of Missouri-Columbia*



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# CONTROLLING SLAB CRACKING

**By Richard B. Heagler, P.E.**

CRACKS OCCURRING IN COMPOSITE DECK FLOOR SLABS IN BUILDINGS do not generally represent a structural problem. The most common cracks are those over beams (or girders) at column lines, while cracks over interior floor beams are less common. Deck manufacturers assume the concrete will crack over every beam and publish live load tables for single span composite slabs.

When asked about the effect of slab cracking on composite beam behavior, the late Dr. Roger Slutter from Lehigh University pointed out that cracking along the beam occurred at an early stage in every composite beam test and, therefore, the composite beam design formulas were based on a cracked system. However, while concrete cracking is not a structural concern under most loading conditions, it can be an appearance problem if carpeting or some other covering is not going to be used.

The Steel Deck Institute (SDI) makes the comment: "If welded wire fabric is used with a steel area given by [0.00075 times the concrete area above the deck flutes] it will generally not be sufficient to be the total negative reinforcement; however, the mesh does a good job of crack control especially if kept near the top of the slab (3/4 in. to 1 in. cover)." The 0.00075 rule is based on experience with mesh as the temperature reinforcement. Other experience factors, such as 0.002 times the area (the ACI requirement for flat slabs), are also used. Again, 0.002 reinforcement will not, in most cases, furnish enough steel to develop the full negative moment capacity.

Reinforcing to control cracking is still primarily a judgement call. In my opinion, the amount of reinforcing steel (for slabs over beams at column lines) should be somewhere between the amount needed to develop the negative moment caused by the design load and the 0.00075 factor of the SDI. The 0.002 factor is in this range and some designers may prefer this as a general rule of thumb. Special loading cases, such as cantilever and moving concentrated loads, do require full negative steel, though.

For transverse reinforcement over girders, where the deck flutes are parallel to the structural steel, the 0.00075 factor should be sufficient for most cases. The position of the reinforcing steel is probably more important than the amount used, and it is necessary that the steel be located near the top of the slab.

*Richard B. Heagler, P.E., is Director of Engineering with Nicholas J. Bouras, Inc.*

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Grid Reinforced Bridge Decks, comprised of both a fabricated steel grid and concrete, are lighter than traditionally reinforced decks and are still strong enough to withstand high traffic volumes over long periods of time (some applications are already in their sixth decade of service). The Bridge Grid Flooring Manufacturers Association maintains a computerized data base of grid related research and welcomes inquiries. The association can provide design recommendations and also publishes a newsletter.

For more information, contact: BGFMA, 231 South Church St., Mt. Pleasant, PA 15666 (412) 54 7-2660 or circle no. **87** on the reader service card near the back of this magazine.

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The Steel Deck Institute offers an expert design system, based on LRFD, for composite and non-composite beams and girders with steel deck. This software is part of the design Advisor expert system developed for the AISC and the SDI by Structural Engineers, Inc., of Radford, VA. Complete bay design as well as individual beams and girders can be investigated and optimized for the least cost. Design tables in the SDI format can be produced using any combination of material properties. Detailed reports are produced showing vibration analysis and provide stud spacing. Concentrated loads and line loads can be applied in addition to uniform loading. Cost is $295.

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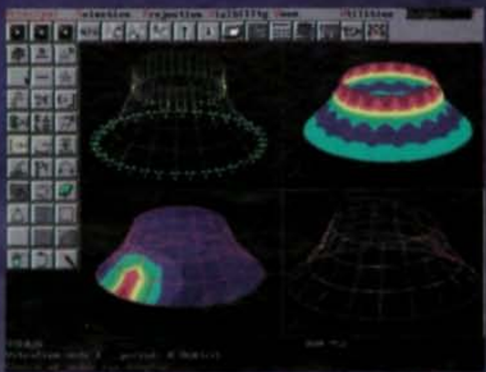


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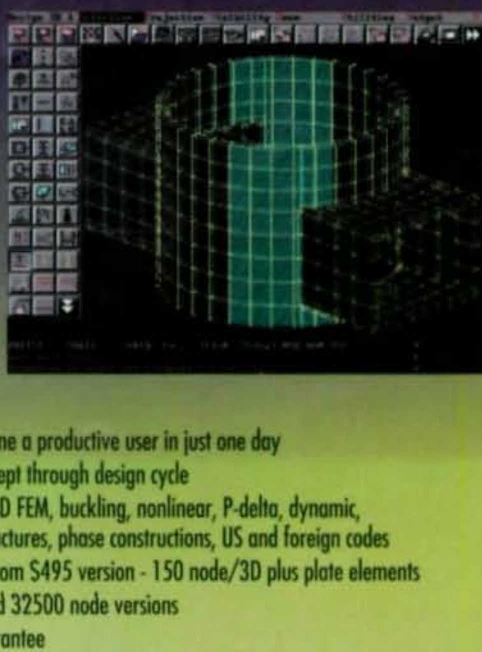

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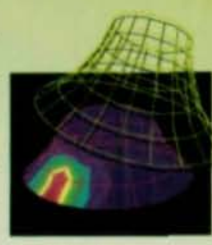

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