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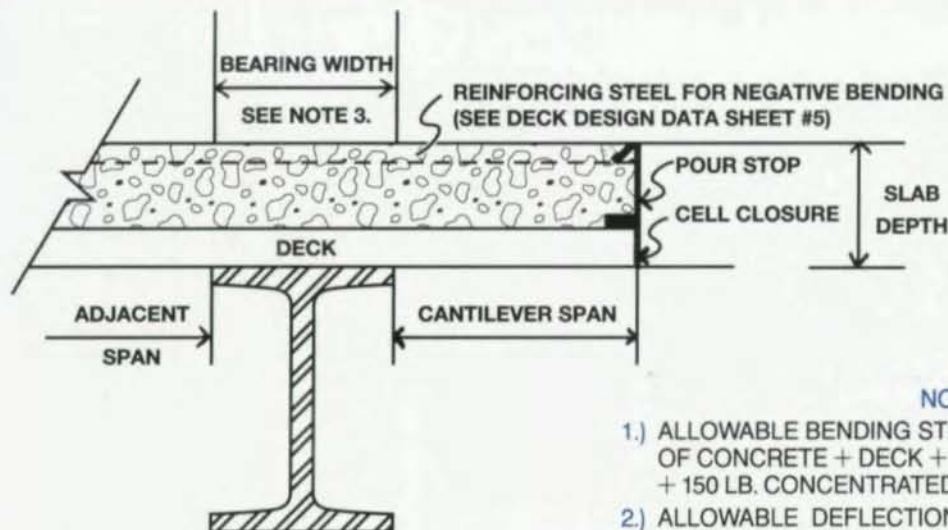
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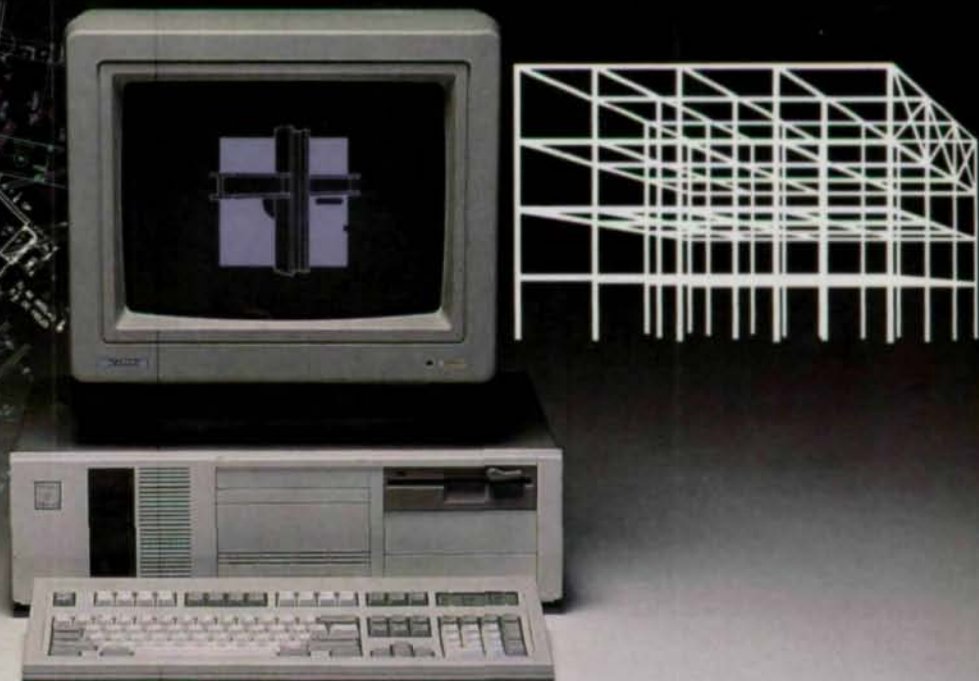
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MODERN STEEL CONSTRUCTION

Volume 31, Number 6

June 1991



Memphis' newest arena is a 450'-square, 290'-tall pyramid with a 52° angle of inclination. The unusual shape required careful analysis and construction planning. The story behind this wondrous structure begins on page 28.

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Lately, everywhere I turn I notice demolition activity. Driving on Chicago's south side I see old Comiskey Park being torn down; during a presentation last week on the Loma Prieta earthquake, I saw the remains of the Nimitz Freeway; and on a recent trip to Detroit I couldn't help but notice several buildings being torn down.

The sight of a wrecking ball is at once awe inspiring and heart wrenching. Regardless of how progress-oriented we are as a society, most people still have a soft spot in their heart for the past.

But nostalgia aside, there's another reason to worry about the destruction of old buildings: The Environment. Today we need to worry not only about preserving the past, but also ensuring the future. And in this day and age of rapidly filling landfills, owners are faced with the problem of what to do with old building debris.

The problem is especially acute with a concrete structure. Concrete columns, beams and girders cannot be reused for much of anything except landfill—and as we're all away by now, that's one thing of which we don't need any more. (How much space does the destruction debris of a 1¼-mile long elevated roadway such as the Nimitz Freeway fill?)

In contrast, when an owner takes down a steel structure, the structural steel members can be recycled—melted down and reformed into new members for use on future projects (there was even a case a few years ago of a steel parking structure being dismantled and erected on a different site). And in addition to the ecological benefits of reducing waste, recycling saves money. The cost of the demolition contract is reduced by the value of the steel scrap.

Still another advantage is that steel structures often don't have to be demolished at all. If an owner needs a larger building, steel structures can often be easily expanded, in which case the demolition waste production is kept to an absolute minimum. Likewise, if a bridge needs to be modified to accommodate increased traffic, steel is the ideal material.

So the next time your kid or a neighbor asks what you've done lately to help the environment, tell them you designed a steel structure. **SM**



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Introducing A Deeper Domestic I-Beam

A 40"-deep, structural wide-flange I-beam is now available from Bethlehem Steel Corp. The W40 structural beam, which is used primarily for short-span steel bridges, is the deepest domestically-produced wide-flange shape.

The company's first 40"-beam had 12"-wide flanges and weighed 167 lbs. per ft. Currently, Bethlehem is concentrating on the lighter weight sections (149, 167, 183, and 211 lbs. per ft. of length). Sections will be available in ASTM Grades A36, A572-50, and A588 Grade B.

"For now, we're going to concentrate on the lighter section weights, and once we get that accomplished we'll work in the heavier sections," said Andrew R. Futchko, manager of operations at

Bethlehem's Structural Products Division.

Deeper rolled shapes are important to various segments of the construction industry. The alternative is steel plates welded to form a beam of the appropriate size and strength.

"The availability of W40s will accelerate our structural customers' ability to accomplish construction work in more cost-effective ways," explained Jeff Manty, a Bethlehem mill superintendent. The availability of W40 beams will primarily affect the construction of short-span bridges.

"A bridge fabricator, for example, could save considerable time and expense by not having to fabricate built-up sections from three



separate pieces of steel," explained Robert E. Roll, Bethlehem's manager of the Structural Products Division's sales and marketing group. "Readily available rolled W40 beams would eliminate those higher costs and related shop fabrication times in many cases."

Book Review: Another LRFD Text Makes The Scene

By Robert F. Lorenz

A new LRFD textbook covering Load and Resistance Factor Design is now available for your technical bookshelf. "Structural Steel Design—LRFD Approach" was written by J.C. Smith, a professor of civil engineering at North Carolina State University. While much of the text serves as a primer, the author does introduce several innovative concepts.

The author's thrust is basically student-oriented. The text offers reminders and tips to the reader about various assumptions that are made, particularly in the approach to problems. While this may be a distraction to the more experienced practitioner, it no doubt serves less experienced readers well.

A close reading of the text does reveal some innovative solutions to problems, however. I would particularly recommend the chapter on bracing where the author thoughtfully blends the AISC limits for out-

of-plumb and initial crookedness of compression members into a variety of bracing stability models.

Another innovation is the use of a novel lower bound treatment to inelastic column buckling problems, which is usually ignored in design practice.

The book's layout features very helpful decimal chapter divisions, which greatly eases a reader trying to locate specific topics. The book features 106 design examples and 198 problems distributed through its 11 chapters. One notable omission, though, is the use of photographs of steel structures that could serve to remind the reader what their designs should finally look like. Fortunately, Smith does make up for this to some extent with bold, eye-catching figures and sketches. Still, many traditionalists will miss photos, especially those of monumental structures such as the Golden Gate Bridge and the St. Louis Arch.

Since LRFD is relatively new, this treatise has a pioneering characteristic as it plows the virgin fields of LRFD utilization. I would recommend this book for some of the unique approaches to problem-solving. Also, some readers will particularly like the author's treatment of braced and unbraced frames.

Although experienced designers may prefer a more tightly written text, the abundance of explanatory material aimed at students shouldn't stop anyone from searching the book for the large number of new techniques described in its 570 pages.

To order the \$57.95 book, call either John Wiley & Sons, Inc., at (201) 469-4400 or AISC Publications at (312) 670-2400 extension 433.

Robert F. Lorenz is director of education and training for the American Institute of Steel Construction, Inc.

Dear Editor:

The March 1991 article titled "Bidding Alternate Design For Bridge Construction" by Robert J. Desjardins (page 17) pointed out issues that we in Florida have been dealing with for some time.

Florida has been involved with segmental bridge design and construction for many years. No doubt some of Mr. Desjardins' experience relates to some of our bridges, however, we have seen similar problems regardless of bridge type. For example:

1. Evaluating Alternates Through Cost.

We have long been a proponent of including costs such as design, design review, CEI, shop drawings reviews, and life cycle in evaluating alternates. However, isolating costs attributable to potential cost overruns due to delays and claims may be very difficult. One prob-

lem is in isolating the cause to specific design issues attributable to a bridge type. These costs seem to occur regardless of the bridge type although, as the article points out, they may be more prevalent with one type than with others.

2. Evaluating Alternate Through Design.

Trying to compare alternate designs is a difficult task under the best conditions. Whether or not the work is done by the same designer or by various designers, maintaining an objective view is difficult and is compounded by introducing different materials.

To address this problem, the FDOT developed the Bridge Development Report (BDR) phase. Basically this has traditionally been "conceptual study", "type, size, and location" phase, but taken to a more comprehensive level. Within the BDR's framework, we thoroughly explore all viable alternates

allowing us to make a reasonable comparison and a rational selection of alternates. The BDR looks at economical span lengths vs. superstructure types; comparative substructure types (piles, drilled shafts, etc.); aesthetics; life cycle costs and other major issues impacting final design. The end result is a document clearly defining all major issues and the rationale of each decision. Once approved by all the parties involved, both at District, State, and FHWA level, the document establishes the scope of the final design phase of the project.

As expected, the BDR's usefulness is directly proportional to the project's importance. When used properly, all parties are forced to make major decisions early, a comprehensive design scope of services can be written, the designer can schedule his work efficiently, and we are able to control the de-

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velopment of the alternates thus assuring as equitable a competition as possible.

3. Evaluating Alternate Through Reviews.

The other means of achieving equitable alternates is through proper reviews of the design plans. This, however, requires time and experienced personnel. There is a common truism that the more expensive and critical the project, the less time there is to design it and schedules, or the need to meet pre-set submittal dates, becomes a significant driving force. As a result, reviewers are often pressured to finish as quickly as possible. For states fortunate enough to have either a highly centralized operation and/or that do a significant proportion of their own design, the problem may not be so acute. In Florida, however where we have both a semi-decentralized operation and do about 90% consultant

bridge design, reviews become a sensitive issue. The consultant is driven by a need to meet the submittal dates (that's how he'll be graded and paid) thus the plans submitted may not be as complete as required and the reviewer is put on the spot.

Another argument used to reduce review time is that the savings due to the efficient design are often less than the expense caused by delaying the letting (i.e., cost of money, increased user costs, etc.). The argument is often posed in the form: What will the more efficient design save vs. what will the delay cost? In effect, the reviewer is asked to weigh structural efficiency for project expediency.

Manpower, another component of the review equation, may also be limited both in number and experience. The Florida DOT is fortunate in having a highly experienced structures staff that can review and

design almost every type of structure type we have. Our problem is that there is always a manpower shortage with the potential of further reductions. In fact, the most common scenario is: reduce manpower, increase duties, and demand faster results.

4. Design Competitions.

Two possibilities are "design/build" and "design competition". In Florida, we recently built a bridge under the design/build concept with variable results. The primary criticism is that a consultant cannot afford the expense of doing the design at the risk of not getting the job. The contractor, on the other hand, may have the financial backing but still sees little reason to take the risk. A compromise may be to pay the design/build teams a nominal amount that will cover some of the expense.

As with the design/build issues,

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a true design competition incurs costs that can be prohibitive and some mechanism to offset such costs must be used to attract truly qualified and capable competitors. Design competitions have been used recently in Maryland and Washington, DC. We explored the possibility in Florida with a large bridge project in Jacksonville but found that state statutes do not permit this kind of competition.

5. Constructibility Reviews.

We wholeheartedly agree with this effort and have tried this process on several projects but found several impediments. First is the time restraint mentioned earlier. If done in-house, we can control and adjust schedules accordingly. If we go outside to either an individual contractor or to the contracting industry, we have little control on time. Secondly, going to a contractor introduces the problem of allowing a few contractors the opportunity to see a bridge project before other qualified and interested contractors. The second problem needs to be addressed by industry organizations.

In conclusion, the issues pointed out are valid and certainly need to be looked at closely. From our perspective, however, there are two additional issues. First, the amount of review time by both the designer (consultant) and the reviewer needs to be given as much importance as the design itself. Secondly, the very nature of our litigious society creates a highly defensive attitude in all participants.

We may not be able to do much about the second, but we should be able to do something about all the other issues.

Antonio M. Garcia, P.E.
Chief Structures Engineer
Florida Department of
Transportation

Dear Editor:

I read with great interest the article in your January issue regarding the electronic transfer of data between the engineer and the fabri-

cator (page 53). This subject is one which is near and dear to my heart since I have devoted my career to creating solutions to these problems. However, in my humble opinion, although your facts were correct, your conclusions ignore important possibilities which are or soon will be available to us.

Over the last decades, responsibility for accurate design has shifted more and more from the

Such a trend is
 unhealthy, costly,
 and irresponsible

engineer to the fabricator and detailer. Engineers have chosen to accept less responsibility for their designs, leaving more details to be worked out by the fabricator.

Ultimately, this trend will lead to engineers who simply draw a conceptual drawing of what a building or plant is to look like and leave it entirely up to the fabricator to design and build. Such a trend is unhealthy, costly, and irresponsible.

As a result, great expense is involved in bidding a project, since the fabricator must conceptualize substantial portions of the project in order to formulate a safe bid. The less detailed the bid documents, the greater the "fudge factor" which must be built in to cover unforeseen circumstance. Since for every successful bidder there are several who are unsuccessful, the overall cost of the estimating process is higher than it should be. The customer ultimately picks up the tab for a lot of unnecessary overhead.

A better approach would be for the engineer to completely design the structure, including all details. He should provide the fabricators with bid documents including an electronic bill of materials. The fabricator would apply his labor and material costs using computerized

estimating tools. The result would be that all bidders are comparing "apples to apples" with greatly reduced risk of omissions. Design responsibility would return to the engineer, who is getting paid to design the bloody thing in the first place.

This reversal of the trend will be implemented by improved technology. Companies like Steelcad International are nearly ready to deliver products which allow the engineer to "stick frame" a building and then turn that into detailed shop drawings electronically. Steelcad and other design programs are or can be smoothly integrated to production control software, like our Fab/Trol System.

The engineer will conceptualize the structure, automatically creating a detailed Bill of Materials which can be electronically transferred to software like ours for creation of estimates, purchase orders, cut lists, inventory reduction, production control and bills of lading. Responsibility is properly assigned to the parties best prepared to accept it. The ultimate recipient will be the customer.

Douglas Cochran
Director of Development
Fab/Trol Systems, Inc.
Eugene, OR

Dear Editor:

We appreciate the fine article on the Delta Airlines Orlando Flight Center (page 12) in the April 1991 edition of *Modern Steel Construction*. The layout use of photographs was done extremely well and portrays the building in its true light.

However, one misconception occurred in the text. ACI/Mitchell was the general contractor for the steel construction package but Great Southwest Corp. was the general contractor for the architectural and the MEP package.

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and minimum out-of-service time. The modules can be placed either longitudinally or transversely. Their light-weight design characteristics make Inverset the ideal solution where dead load is a constraint.

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Weathering steel used in Bridge System? and the short of it.



Bethlehem's weathering steel was recently used by The Fort Miller Co. in two upstate New York projects. The Rockwell Falls Bridge rehabilitation project for the Hudson River Bridge at Lake Luzerne, NY, required just 28 days of downtime. The units, which were positioned transversely, replaced the existing floor beams and deck on the 180-ft.-long bridge.

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Bethlehem Steel Corporation, Construction Marketing Division, Bethlehem, PA 18016.

Rockwell Falls Bridge: (left)

Owners: Saratoga and Warren Counties, State of New York

Fabricator: Seibel Modern Manufacturing and Welding Co., Lancaster, NY

General Contractor: J. H. Maloy, Inc., Albany, NY

Outlet Road Bridge: (right)

Owners: Saratoga County, State of New York

Fabricator: Seibel Modern Manufacturing and Welding Co., Lancaster, NY

General Contractor: Schultz Construction Inc., Round Lake, NY

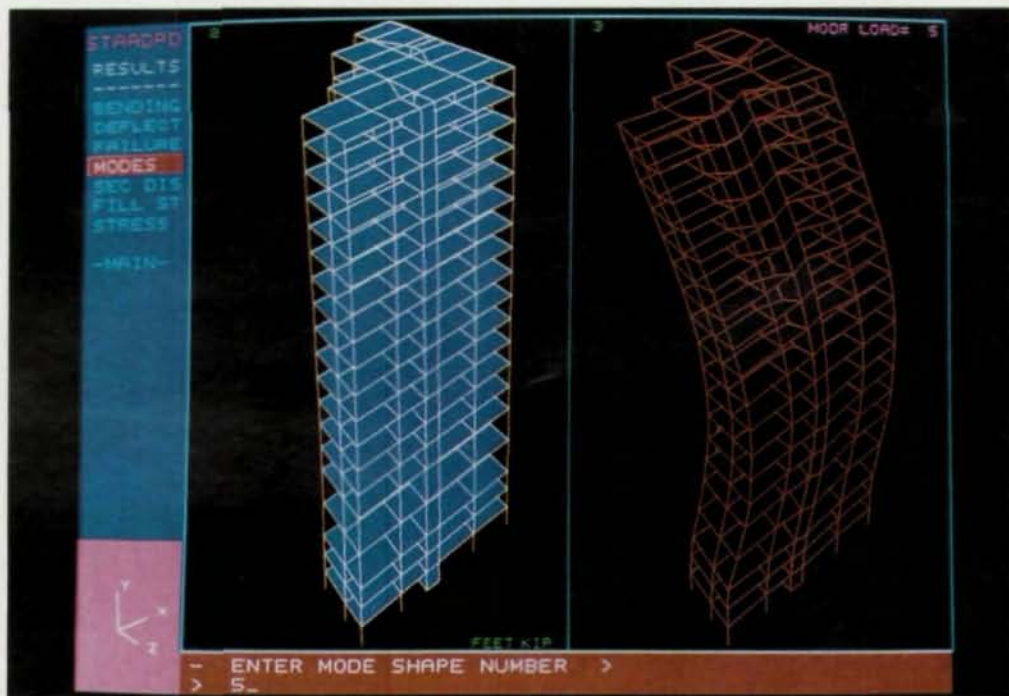
Consulting Engineer: Greenman-Pedersen Inc., Albany, NY

Inverset Licensee: The Fort Miller Company, Inc., Schuylerville, NY

Weathering Steel Supplier: Bethlehem Steel Corporation, Bethlehem, PA

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Controlling Floor Movement

Serviceability problems can be controlled by the proper engineering of damping and the frequency of structural members



Thomas M. Murray, Montague-Betts Professor of Structural Steel Design at Virginia Polytechnic Institute, is the winner of this year's AISC T.R. Higgins Lectureship Award. Murray will present his paper at least seven times in 1991-92, beginning at this year's National Steel Construction Conference in Washington, DC. Cities where the paper will be presented include: Chicago; St. Louis; Kansas City; Tucson; Los Angeles; and Dallas. For more information, write: Robert Lorenz, Director of Education, American Institute of Steel Construction, Inc., One East Wacker Dr., Suite 3100, Chicago, IL 60601-2001.

"Annoying floor motion induced by building occupants is probably the most persistent floor serviceability problem encountered by designers," according to this year's AISC T.R. Higgins Lectureship Award winner.

Thomas M. Murray, Montague-Betts Professor of Structural Steel Design at Virginia Polytechnic Institute and State University, will present his paper, *Building Floor Vibrations*, at six venues this year, beginning at this year's National Steel Construction Conference. The paper presents an overview of analytical tools and concepts for controlling annoying floor movement in residential, office, commercial and gymnasium environments.

"A number of procedures have been developed by researchers that allow a structural designer to analytically determine occupant acceptability of a proposed floor system," Murray writes. "Generally, the analytical procedures require the calculation of the first natural frequency of the floor system and either maximum amplitude, velocity, or acceleration of a reference excitation. An estimate of the damping in the floor system also is required in some instances. A human perceptibility scale is then used to determine if the floor system meets serviceability requirements.

Residential And Office Design

In residences and offices, vibra-

tions occur primarily as a result of either individuals or small groups walking across the floor. "As a result," Murray states, "the most important parameter for residential and office environments is damping."

In his paper, Murray states: "If the following inequality is satisfied, motion of the floor system caused by normal human activity in office or residential environments will not be objectionable to occupants: $D \geq 35 A_0 f + 2.5$ where D = damping in percent of critical, A_0 = maximum initial amplitude of the floor system due to a heel-drop excitation (in.) and f = first natural frequency of the floor system (hz).

The heel-drop excitation used to develop the criterion can be approximated by a linear decreasing ramp function having a magnitude of 600 lbs. and a duration of 50 milliseconds.

"The criterion was developed using a field measurements of approximately 100 floor systems mostly in the frequency range of 5-8 hz," Murray said. "Use of the criterion for floor systems with a natural frequency above about 10 hz is not recommended."

Murray warns, however, that "Use of this criterion requires careful judgement on the part of the designer."

For example, while a typical office building floor system with hung ceiling and minimal mechanical ductwork exhibits about 3% of

critical damping, additional damping may be provided by office furniture, partitions, equipment and the occupants themselves. "If the required damping is less than 3-3.5%, the system will be satisfactory even if the supported areas are completely free of fixed partitions." However, when the required damping is between 3.5% and 4.5%, the final configuration and its intended use must be carefully considered.

And if the required damping is much greater than 4.5%, the designer must either identify an exact source of damping, provide additional damping, or redesign the space. Framed-in-place partitions are very effective sources of damping when each partition is attached to the floor system in at least three locations and they are located within the effective beam

spacing or the effective floor width that is used to calculate system amplitude, Murray said.

When partitions are not used, an inexpensive method of artificially increasing the damping is to install "false" sheetrock partitions of maximum depth in the space between

those below about 3 hz and those between 5-8 hz. In more than 50 floor problems investigated by Murray, the problem has been where the measured first natural frequency of the floor system is between 5 and 8 hz. Murray said he has never encountered a problem with floor spans greater than 40', which is contrary to the common belief that long span floor vibrate and should be avoided. "Furthermore, an office/residential floor with a natural frequency greater than 10 hz has never been found to be a problem."

Framed-in-place partitions are very effective sources of damping when each partition is attached to the floor system in at least three locations

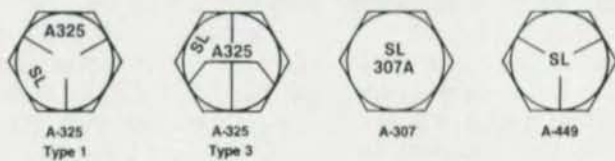
the ceiling and the underside of the floor slab. Murray states that his tests have shown an increase in damping of 0.5% to 1% using 2'- to 3'-deep false partitions.

Another floor vibration problem deals with frequencies, especially

Retail Design

Because the forcing function in retail settings is nearly continuous walking or running movement, damping is not as critical a concern as it is for office/residential environments. "Control of the stiffness

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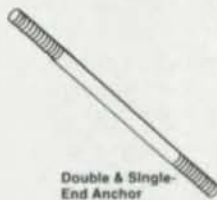
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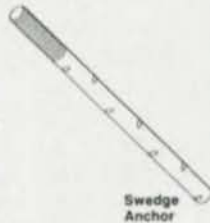
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of the structural system is the best solution," Murray states.

Murray cites Bruce Ellingwood, a professor at Johns Hopkins University and a former Higgins Award winner, who recommends a criterion based on an acceleration tolerance limit of 0.005 g and walking excitation. "This criterion is satisfied if the maximum deflection under a 450 lbs. (2 kN) force applied anywhere on the floor system does not exceed 0.02 in.," Murray said.

Gymnasiums

"For floor systems supporting dancing or exercise activities, damping is usually not of consequence," according to Murray. Because the accompanying music for aerobic activity does not exceed 150 beats per minute, the resulting forcing frequency is only about 2.5 Hz. However, the first natural frequency of floors supporting such activity should be above 7-9 Hz to

avoid resonance.

To avoid floor-motion complaints, Murray recommends: providing structural framing so that the first natural frequency is generally above 9-10 Hz; isolating the floor systems from the remaining structure using separate columns; separating ceilings and partitions immediately below the exercise floor by supporting the ceiling on its own framing and by not extending partitions to the floor above; and accepting the possibility of complaints from non-participating individuals who happen to be on the exercise floor during activity by medium to large groups (20 to 60 participants).

According to Murray, the least expensive method of obtaining structural framing with sufficient stiffness to meet the 9-10 Hz criterion is to use deep beams or joists and lightweight concrete slabs (a decrease in mass increases frequency).

Murray's full paper, *Building Floor Vibrations*, will be printed in the *Proceedings of the 1991 National Steel Construction Conference* and in the *Third Quarter 1991 Issue* (available in September) of *Engineering Journal*.

Copies of the *Proceedings* can be obtained for \$35 + tax + UPS shipping charges by contacting: AISC, Publications Department, P.O. Box 806276, Chicago, IL 60680-4124. For phone orders, call (312) 670-2400 ext. 433.

Single issues of *Engineering Journal* are available for \$5 (\$6 foreign) by writing to: AISC, *Engineering Journal*, P.O. Box 806276, Chicago, IL 60680-4124. One year subscriptions are \$15 (\$18 foreign surface mail; \$44 air mail) and a three year subscription is \$36 (\$45 foreign surface mail). No phone orders are accepted for *Engineering Journal*. □

AISC SOFTWARE

LRFD/ASD Computer Software Data Base For Structural Shapes

The AISC Computer Data Base contains properties and dimensions of structural steel shapes corresponding to data published in Part I of the *1st Edition, LRFD Manual of Steel Construction* as well as the *9th Edition, ASD Manual of Steel Construction*.

The program includes the Computer Data Base in ASCII format for the properties and dimensions of the following shapes: W Shapes; S Shapes; M Shapes; HP Shapes; American Standard Channels (C); Miscellaneous Channels (MC); Structural Tees cut from W, M and S Shapes (WT, MT, ST); Single & Double Angles; and Structural Tubing and Pipe.

Steel Member Fire Protection Computer Program

STEMFIRE determines safe and economic fire protection for steel beams, columns, and trusses. It is intended for use by architects, engineers, building code and fire officials, and others interested in steel building fire protection. The software data base contains all the pertinent steel shapes and many listed UL *Fire Resistance Directory* construction details and their fire ratings. In this manner, user search time is minimized and the design or checking of steel fire protection is optimized.

Steel Connection Design Software (CONXPRT)

CONXPRT is a knowledge-based PC software system for the design of steel building connections. Three basic types of connections are included in Version 1.0: double framing angles; shear end-plates; and single-plate shear connections. More than 80 configurations are possible.

All designs are according to procedures in the AISC *9th Edition* (ASD) or latest available references. CONXPRT includes complete data bases for standard shapes, the structural steel, weld and bolt materials listed in the *9th Edition*. All strength and serviceability limit states and dimensional requirements for each design are checked. Help menus are included.

WEBOPEN

This state-of-the-art software package is based on and includes the new AISC *Design of Steel and Composite Beams with Web Openings*. The program is designed to enable engineers to quickly and economically design beam web openings. The easy-to-use color coded input windows provide a clear, logical data entry system.

WEBOPEN was written by practicing engineers and incorporates "expert" design checks and warning messages that enhance the application of the AISC Design Guide to your design problems.

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For more information, contact: AISC Software, One East Wacker Dr., Suite 3100, Chicago IL 60601-2001 (312) 670-5434.

IN A RECESSION, THE BEST DEFENSE IS A GOOD OFFENSE.

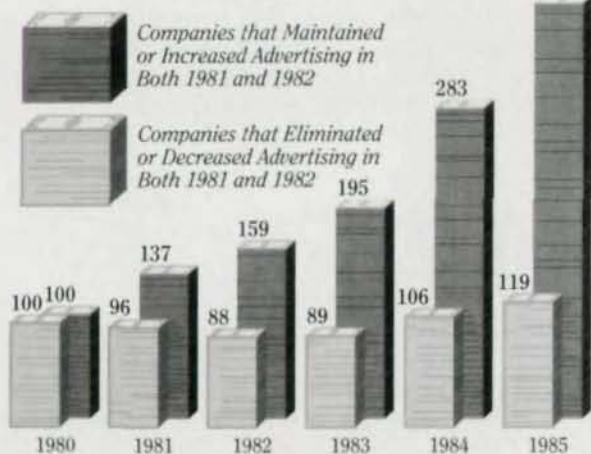
It's a recession. Your instincts demand that you cut the ad budget. But, as the McGraw-Hill Research¹ analysis of business-to-business advertising expenditures during the 1981-82 recession shows, it's those with the courage to maintain or increase advertising in a recession who reap a major sales advantage over their competitors who panic

and fall back into a defensive posture. And this advantage continues to expand long after the recession is over.

Recessions last an average of 11 months, but any advertising decision made during one can have permanent repercussions. The McGraw-Hill study demonstrates that nervous advertisers lose ground to the brave and can't gain it back. In 1980, according to the chart seen here, sales indices were identical, but by 1985 the brave had racked up a 3.2 to 1 sales advantage. A similar study done by McGraw-Hill during the 1974-75 recession corroborates the 1980's research.

A recession is the single greatest period in which to make short- and long-term gains. And, surprisingly, increasing advertising modestly during one has much the same effect on your

Effects of Advertising in a Recession on Sales (Indices)



McGraw-Hill Research, 1986.

profits as cutting advertising does. According to The Center for Research & Development's October 1990 study of consumer advertising during a recession, advertisers who yield "to the natural inclination to cut spending in an effort to increase profits in a recession find that it doesn't work."² This study, relying on the PIMS³ database, also uncovered that aggressive recessionary advertisers picked up 4.5 times as much market share gain as their overcautious competitors, leaving them in a far better position to exploit the inevitable recovery and expansion.

Chevrolet countered its competitors during the 1974-75 recession by aggressively beefing up its ad spending and attained a two percent market share increase. Today, two share points in the automotive industry are worth over \$4 billion. Delta Airlines and Revlon also boosted ad spending in the 1974-75 recession and achieved similar results.

Continuous advertising sustains market leadership. And it's far easier to sustain momentum than it is to start it up again. Consider this list of market category leaders: Campbell's, Coca-Cola, Ivory, Kellogg, Kodak, Lipton and Wrigley. This is the leadership list for 1925. And 1990. These marketers have maintained a relentless commitment to their brands in both good times and bad. Kellogg had the guts to pump up its ad spending during the Great Depression and cemented a market leadership it has yet to relinquish.

These are the success stories. Space and diplomacy don't allow the mention of the names of those who lacked gusto and chose to cut their ad spending in recessionary times.

But if you would like to learn more about how advertising can help make the worst of times the best of times, please write to Department C, American Association of Advertising Agencies, 666 Third Avenue, New York, New York 10017, enclosing a check for five dollars. You will receive a booklet covering the pertinent research done on all the U.S. recessions since 1923. Please allow 4 to 6 weeks for delivery.

¹ McGraw-Hill Research, 1986. ² The Center for Research and Development ©1990. ³ Profit Impact of Market Strategies, The Strategic Planning Institute, Cambridge, MA.

AAA

Reducing Airport Congestion

A new steel elevated roadway system at New York's JFK International Airport is expected to ease access for its annual crunch of 30 million passengers

By Oscar Suros, P.E., and Herbert Y. Chu, P.E.

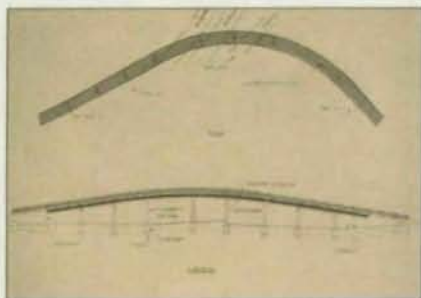
When it opened in 1947, John F. Kennedy International Airport in New York City was designed to serve an estimated 15 million passengers a year. Today, more than 30 million passengers annually use the airport. And by the year 2000, that number is expected to climb to 45 million.

In response, The Port Authority of New York and New Jersey is undertaking a major redevelopment program. One of its main features is a complex roadway network consisting of about 250,000 sq. ft. of elevated structure. As the airport has to be fully operational during the construction period, the selection of a suitable structural scheme was challenging.

The airport has eight passenger terminals and an International Arrivals Building complex. More than 5,000 vehicles per hour congest the airport's roads and all ground traffic uses one of two expressways for



The new elevated roadway at John F. Kennedy International Airport in New York City was designed to easily handle the more than 5,000 vehicles arriving at the airport every hour.



entering or exiting the airport. Both inadequate passenger drop-off and pick-up points and circuitous routing of vehicles contribute to traffic problems.

The new roadways are designed to provide easier access to the Central Terminal Area, the airline terminals, and the International Arrivals Building.

Functional Requirements

The major geometric features of the roadway system include:

- Straight alignment with variable roadway width and curved profile;
- Curved alignment with variable roadway width and curved profile superelevation;
- Transition areas (two roadways merging into one or one wide roadway splitting into two) with curved alignment and curved profile.

The structural system selected for this project not only had to ac-

commodate the above geometric requirements, but the designers also had to consider: structural flexibility; constructability; aesthetics; maintenance; and bidder's competition. And, of course, the airport must remain fully operational during construction.

Alternative Structural Systems

Eight steel and concrete superstructure systems were evaluated. To establish a common base for evaluation, a single column pier for the substructure is assumed for all systems.

- Simply Supported Welded Plate Girders: Straight or curved simply supported steel welded plate girders on concealed steel box cap girder.
- Steel Rolled Beams: Straight or curved, simply supported or continuous steel rolled beams over cast-in-place concrete hammer-head pier.

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- **Continuous Welded Plate Girders:** Straight or curved continuous steel welded plate girders framed into concealed, steel I-shaped cap girders.
- **Steel Box Girder:** Straight or curved rectangular steel box girder over cast-in-place concrete hammerhead pier.
- **Adjacent Precast Concrete Box Girders:** Adjacent precast, prestressed concrete straight box girders supported on concealed, cast-in-place concrete hammerhead pier.
- **Precast Concrete AASHTO I-Beams:** Precast, prestressed concrete AASHTO I-beams simply supported on cast-in-place concrete hammerhead pier.
- **Concrete Segmental Box Girder:** Simply supported or continuous precast concrete segmental box girder post-tensioned inside the box, over cast-in-place concrete hammerhead pier.
- **Cast-In-Place Concrete T-Beams:** Simply supported or continuous concrete T-beams, cast-in-place monolithically with the deck, over cast-in-place concrete hammerhead pier.

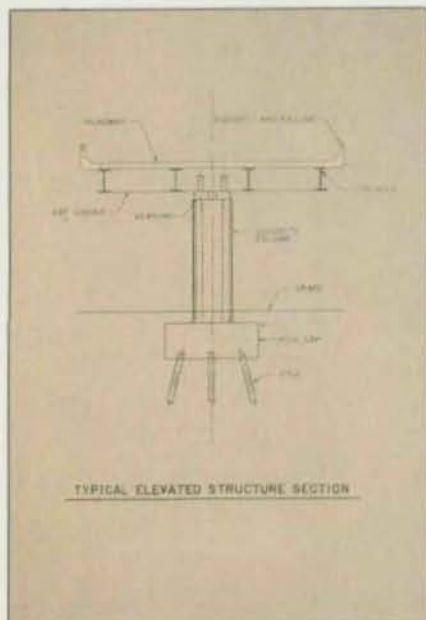
An analysis of the eight systems included structural flexibility, constructability, aesthetics, maintenance, and bidder's competition.

The analysis revealed that all of the concrete structural systems would require greatly reduced span lengths for curved alignment with small radii, which represented approximately 15% of the project. For some systems, cast-in-place construction would be required at transition and at sharply curved areas. As the cast-in-place construction would greatly interfere with airport operations, none of the concrete systems were suitable for this project.

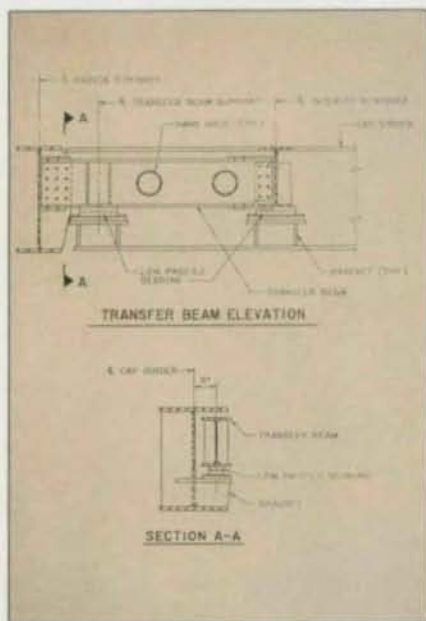
Analysis revealed that the two most desirable systems, both in terms of aesthetics and cost, were both Welded Plate Girder systems. The advantages of both systems include: the ability to handle relatively long spans and curves with various radii; structural depth well-proportioned to corresponding



A continuous welded plate girder system was chosen as being the most desirable in terms of cost and aesthetics, as well as having very low maintenance requirements.



TYPICAL ELEVATED STRUCTURE SECTION



TRANSFER BEAM ELEVATION

SECTION A-A

span length; easy adaptation to the complex geometric requirements at roadway transitions; reduced on-site construction time since all structural steel members are fabricated off-site; relatively simple erection procedure thereby reducing the impact on airport operations; aesthetically pleasing design; and easy repair or replacement of deteriorated deck slabs.

While both systems share the same major advantages, the Continuous Welded Plate Girder system reduces the total number of maintenance problems.

In addition, the simply supported system has a deeper structure for the same span length and requires a raised roadway profile to allow for the required clearance under the structure, thus increasing the total square footage of the elevated roadway structure and the cost of the project. Thus, the Continuous Welded Plate Girder system with steel I-shaped cap girder was selected.

Superstructure Design

The alignment and profile of the roadways are extremely complex due to the limited space and stringent operational restrictions at the airport. About 44% of the roadway structure is on tangent alignment and the remainder on curved alignment. The radii vary from 145' to 10,000'. The maximum cross slope is 6% and the profile has a maximum slope of 6%.

The typical deck structure is an 8½", cast-in-place concrete slab on stay-in-place steel forms. Since the elevated roadway is designed to accommodate vehicular traffic only, there is no sidewalk provided on the deck. To satisfy aesthetic requirements, the fascia plate girder stringer maintains a constant distance of 4' from the face of the deck structure whether it is on a tangent or a curved alignment.

A nominal depth of 3' was selected for all stringers. Since the cap girder is required to be concealed within the depth of the

stringer, and because of geometry and detailing requirements, the depth of the I-shaped cap girders is less than 3'. All stringers frame into the cap girder, except at the expansion joint where stringers are supported on expansion bearings attached to brackets on one side of the cap girder.

Because the depth of the concealed cap girder is less than 3', the maximum width of structure for the single column system is limited to 34'.

For roadway widths exceeding 34', two concrete columns are required to support the cap girder and are placed at locations that do not interfere with other on-grade roadways or major underground utilities. In one instance, three concrete columns are used to support a very long steel cap girder.

To control the size of the deck expansion joint and temperature stresses in the overall structure, a maximum distance of 465' between deck expansion joints is used. Depending upon the location of columns allowed by the on-grade roadway geometry, the span length varies from 48' to 119', and the number of continuous spans varies from two to five.

For the design of the continuous stringers, the Line Girder System program from Bridge Software Development International Ltd. was used for the tangent stringers, and the V-Load Analysis program from United States Steel Corp. was used for all curved stringers.

A 16" and 24" flange width was selected for all stringers and cap girders, respectively, with varying flange thickness controlled by strength, fatigue and deflection requirements. All steel is A572 Grade 50, except for the diaphragms and diaphragm connection plates, which are A36 steel. All field connections are made with 1"-diameter high-strength bolts, except at the diaphragms where ¾"-diameter high-strength bolts are used.

Structural Details

To improve aesthetics, the steel cap girder was concealed to create

visual continuity of the fascia stringer. As a result, at the expansion joint the expansion bearing for the fascia stringer had to be hidden behind the fascia, which is only 3' deep. This was accomplished by using a shallow transfer beam connecting the fascia and the first interior stringer.

To allow temperature movement of the fascia stringer in the longitudinal direction, an expansion bearing attached to the underside of the transfer beam and located 1'-9" from the fascia stringer was provided. For the interior stringer, the expansion bearing is located at the center line of the stringer. All expansion bearings are low profile and seated on shallow brackets welded to the cap girder. All other stringers are rigidly bolted to the cap girder to create a continuous structure.

The connection between the steel cap girder and the concrete column in the single column system resulted in an uplift bearing with two parts: Each has two, 2 3/4"-diameter uplift anchors to develop tension due to the moment effect and seven 1 1/2"-diameter anchor bolts to develop the transverse and longitudinal shears.

Shop-Welded Plates

The sole plate has a curved surface at the bottom to allow the cap girder to roll slightly to accommodate the rotation of the connecting continuous stringers. The sole plate is shop-welded to the cap girder and keyed to the base plate by two, 2 1/4" diameter pintels. Since the rotation of the cap girder is very slight, the uplift anchor connection to the cap girder is capable of accommodating this movement. For cap girders supported on multiple concrete columns, uplift anchors are not required and standard bearings are used.

To minimize field adjustments to the steel framing, the steel fabricator

is required to shop assemble all field connections for a full girder assembly from expansion joint to expansion joint.

As a result, any misalignment or other errors can be corrected prior to transporting the members to the site. Also, to eliminate some common geometry errors during construction, the fabricator is required to survey in the shop the location and orientation of center lines of all cap girders and center lines of cap girder bearing during the shop assembly operation.

Based on airport operational requirements, the project was divided into four phases,

with approximately two thirds of the elevated structure in Phase 1. By April, more than one half of the elevated roadway structures in Phase 1 were complete, and the contractor, Slattery Associates, Inc., Maspeth, NY, had revised the estimated construction time from 35 months down to 25 months, and is expected to be completed by the end of 1991.

Reduced Construction Time

This significant reduction in construction time is mainly attributable to the ease of steel erection in the field and to the quick resolution of field problems between the contractor and the design engineers. The requirement of shop assembling all steel members between expansion joints was an important factor in avoiding field problems and speeding the erection. Steel fabricator for Phase 1 is Harris Structural Steel, Co., South Plainfield, NJ.

Oscar Suros is chief structural engineer and Herbert Y. Chu is principal engineer of The Port Authority of New York and New Jersey. This article was adapted from a paper he presented at the National Steel Construction Conference in Washington, DC. □

Construction time was revised down from 35 months to 25 months

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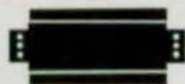
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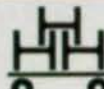
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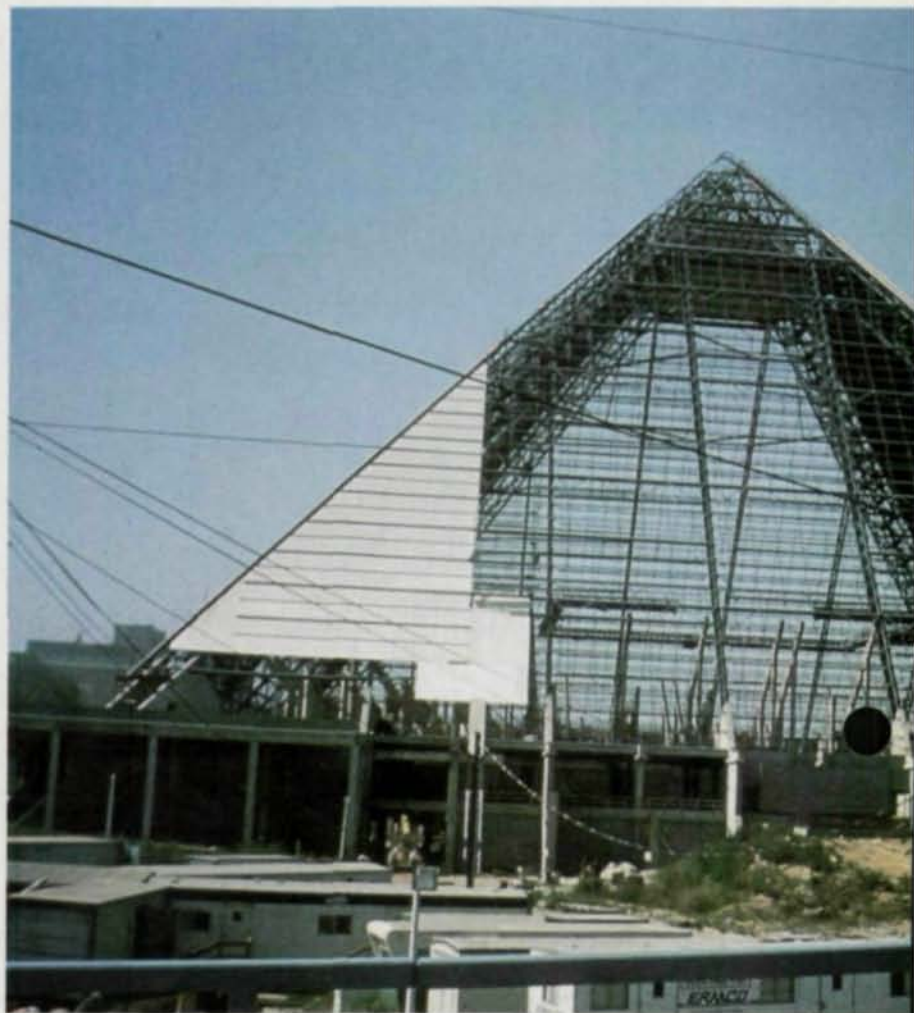
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The Great American Pyramid

A new pyramid is rising in Memphis—but on the banks of the Mississippi River instead of the Nile River

By Lawrence G. Griffis, P.E.



For more than 40 centuries, the pyramids of Egypt have risen majestically above the desert sands of the Nile Valley. And the grandest of all of these is the Great Pyramid of Cheops, which rises 481' and covers 13 acres near the Nile River. The core of the pyramid is made up of natural desert sandstone—approximately 2.3 million blocks weighing a total of 6.5 million tons.

Today, along the banks of another mighty river, the Mississippi, another great pyramid is rising. In stark contrast to its Egyptian fore-runners, however, this pyramid is built of structural steel, reinforced concrete, and stainless steel clad-

ding. Designed by the Atlanta-based architect Rosser Fabrap International with structural engineering by Walter P. Moore and Associates, Houston, the arena is being built on a design/build basis by Indianapolis-based contractor Huber, Hunt and Nichols, Inc., for \$55 million.

The 450'-square, 290'-tall pyramid arena is a 60% scale model of the Egyptian pyramid with the same 52° angle of inclination of its sides. It will be home for the Memphis State University basketball team and have as one of its primary tenants the American Music Hall of Fame. It will also be a multipurpose facility for concerts, the

circus and stage entertainment. It will feature 28 private boxes and an observation deck with a glass skylight at the top.

Site Conditions

The site for the new pyramid is located just west of the downtown area across from the convention center and about 400 yards from the banks of the Mississippi River.

The site and building foundation proved to be one of the bigger challenges facing the design/build team. Extremely heterogenous soil conditions, representing various geologic stages of sedimentation along the river, consisted of a myriad of soil types including sands,



The four corners of the 290'-tall pyramid are framed with box trusses 14' wide and 18' deep. Each box truss is on a massive concrete wall buttress 24" thick and two-stories high.



silts, clays, gravel and the debris of former building foundations.

The structural engineer established a maximum $\frac{1}{2}$ " differential settlement criteria between the concrete buttress supports that support the steel superstructure. The foundation type selected, primarily on the basis of least cost, consisted of approximately 1,200 16"-diameter auger cast piles founded in the Jackson clay formation as deep as 100' from the surface. The stringent design criteria were met using the GROUPI computer program for the battered piles and pile cap design.

Pyramid Superstructure

The sloping walls of the arena

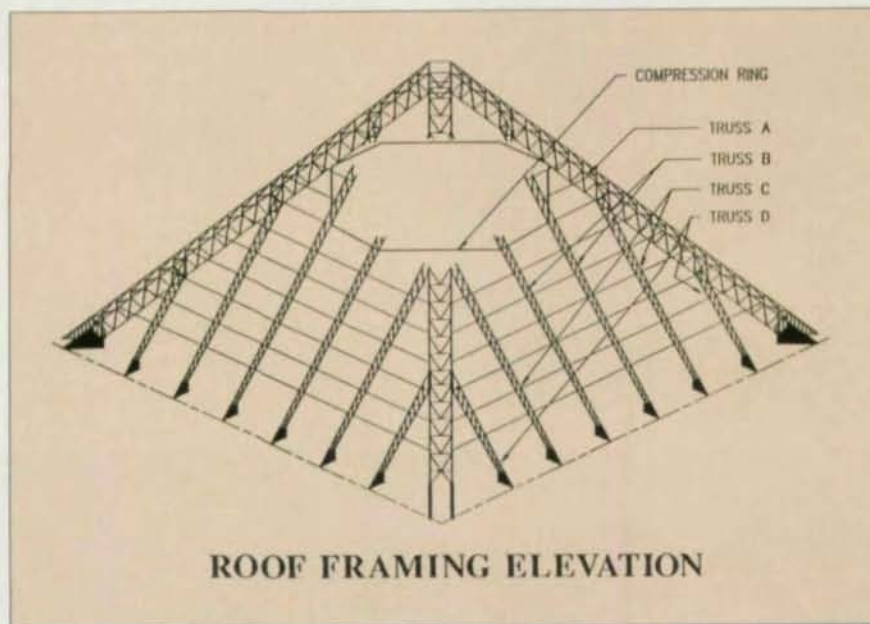
are supported on a system of steel open web joists, beams, and trusses. The four corners are framed with box trusses 14' wide and 18' deep meeting at the apex of the pyramid. Each box truss, consisting of W14 wide flange chord members and double angle web members, weighs approximately 140 tons.

Metal stairs inside two of the four box trusses provide ground level access to the observation level. These structural steel stairs were fabricated and constructed prior to erection. Light steel framing that supports the outside cladding of the ridge above the trusses also was constructed in place on

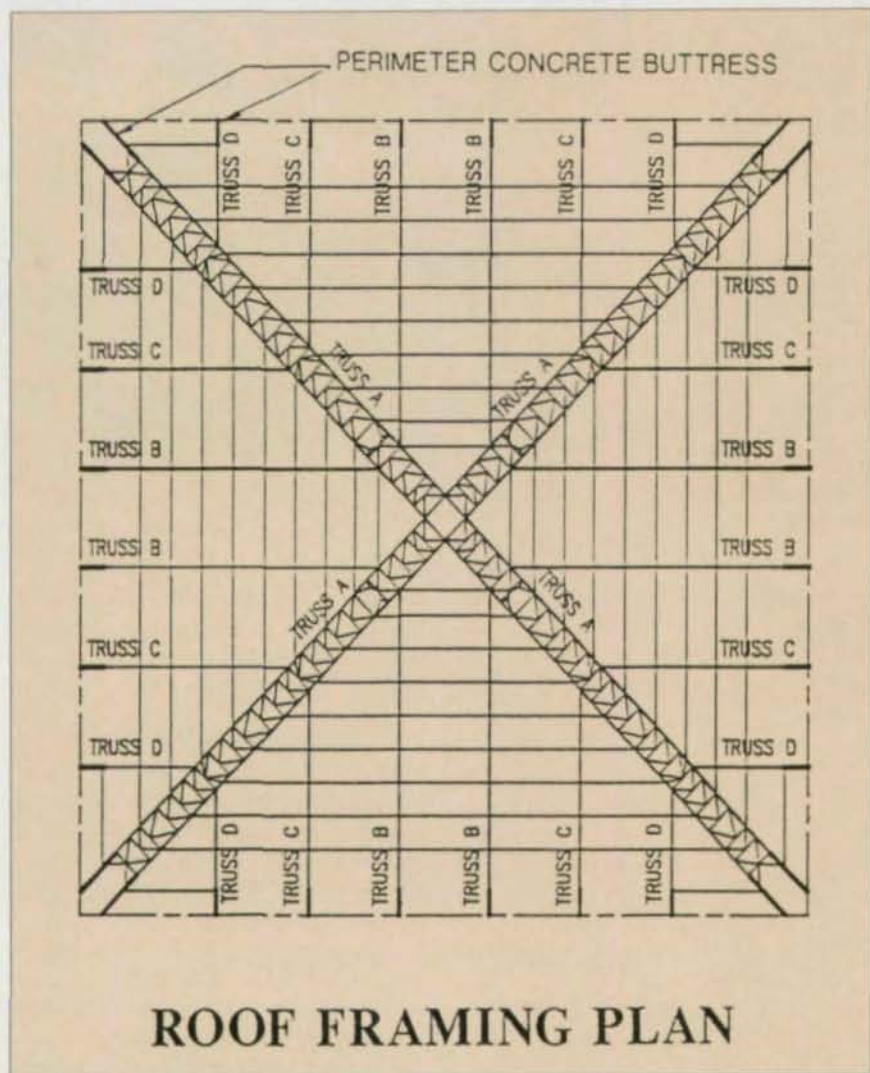
the ground, making the heaviest box truss lift about 200 tons. Each box truss is on a massive concrete wall buttress 24" thick and two-stories high.

Secondary Trusses

Secondary planar trusses span from additional concrete buttress foundation walls to the box truss above. Steel brace beams are spaced every 30' up the slope and connect to each box truss and secondary truss. The secondary trusses are spaced every 56' across each face of the pyramid. Framing across each pyramid face between the secondary trusses are 32"-deep long-span bar joists. Diagonal bar



ROOF FRAMING ELEVATION



ROOF FRAMING PLAN

joist bridging hold the joists in an inclined plane perpendicular to the face of the cladding.

A total of 24 secondary trusses varying in depth from 6' 6" at the buttress support to 11' 7.2" at the box truss connection. The secondary trusses weigh 11 tons, 26 tons, and 68 tons, depending on size and location. Structural steel weight for the trusses and beams is 1,825 tons, or 18.3 psf of projected roof area. This unit weight is in stark comparison to the 35 psf weight of a flat roof of comparable span (446').

The bar joist and bridging weight is approximately 230 tons. Total weight for all structural steel on the project, including trusses, beams, joists, miscellaneous steel ridge members, observation level, skylight framing, catwalks and rigging steel is approximately 2,550 tons. Structural steel fabricator is AISC-member Havens Steel, Kansas City.

Design Challenges

The superstructure was framed in a manner that took advantage of the inherent stiffness of the pyramid shape. The four corner box trusses behave more as sloping columns than flexural truss elements. The state of stress in these box members is primarily axial because bending is limited by the intersecting trusses acting as beam-columns. These secondary trusses intersect the box trusses in two perpendicular planes and not only brace the box truss but help share the roof loads as well.

The complex roof structure was analyzed using a three-dimensional computer model with the SAP 90 computer program. The model contained all truss and beam members and consisted of 1,818 nodes and 4,187 members. The 198 different possible load combinations of dead load, live load, snow load, wind load and a 75198F differential temperature load were analyzed.

Of particular concern in the design stage was the potential visual effect of structural deflection of the sloping sides and possible rippling

00314

of the stainless steel cladding under load. The potential deflection problem existed because of the different span lengths of the neighboring secondary trusses and the proximity of the maximum deflection point of the B secondary truss to the nearby undeflecting box truss (see diagram). A preliminary computer analysis revealed a 3.4" total deflection of the B truss under dead load.

Economic Design

To adequately stiffen the roof system without spending addi-

Because the pyramid shape is seldom utilized, extensive wind tunnel tests were performed

tional dollars for steel weight, the structural engineer designed a compression ring beam connecting each secondary B truss at its bottom chord level. This ring beam system not only reduced the secondary B truss deflection to 1.4" under dead load but also saved 120 tons of steel weight by reducing the effective length of the B trusses.

The architect and engineer also were concerned about fabrication tolerances, particularly camber perpendicular to the direction of the cladding, and the potential visual effects it would have on the cladding under different lighting conditions. As a result, special fabrication tolerances, rather than those defined by the AISC Code of Standard Practice, were established for the trusses, beams, and bar joists.

Because the pyramid shape is seldom utilized in major modern structures, extensive wind tunnel tests were performed by RWDI, Inc., of Guelph, Ontario, Canada.

The study showed that approximately 90% of the cladding surface will experience wind pressures of 30 psf or less under a 50 year de-



Metal stairs inside two of the four box trusses provide ground level access to the observation level. These structural steel stairs were fabricated and constructed prior to erection.

sign storm. A maximum cladding pressure of 70 psf was identified at a few isolated corners near the base. These pressures proved to be significantly less than the pressures prescribed by the building code for a vertical wall building of comparable height.

RWDI also evaluated snow and ice patterns, and as a result, the architect prescribed less insulation at the ridge cladding and provided heaters in the observation level stairs to prevent ice and snow build-up at these critical locations.

No aspect of construction planning on the project was given more attention than superstructure erection. Because some of the largest stresses induced in the superstructure occurred during erection, the structural engineer prescribed one possible sequence and method of erection on the bid documents and bidders were then allowed to bid the erection using the suggested sequence and lift points shown on the drawings without having to perform a design check. Alternately, they were allowed to bid using a



different erection procedure but only if they included a design check.

Superstructure Erection

The successful erection subcontractor, Barnhart Construction Co., Memphis, elected to erect the superstructure using one central free-standing tower crane under the apex of the pyramid. This tower remained in place until the entire steel superstructure was connected. Two 250 ton cranes and two 150 ton cranes supplemented the center tower crane.

The first A box truss (see diagram) was lifted from lift points near the low end of the span with the two 150 ton cranes and near the upper end with the two 250 ton cranes. The center tower crane lifted the extreme upper end of the box truss. This lift was approximately 200 tons, with each pair of cranes—one 150 ton and one 250 ton—rigged together to insure a 40%/60% load sharing.

The upper cranes held the load while the lower cranes were used to erect the D and C secondary trusses on each side of the box truss.

The procedure was followed in a diagonally opposite quadrant and then in the two remaining quadrants. Subsequent steps involved erection of the B trusses in each quadrant and finally the compression ring beams.

The final stages involved erection of the middle bays on each of the four faces. The erector elected to panelize each 56'-wide by 60'-long (up the slope) bar joist and brace beam bay into a single lift. These panelized bays were erected with the secondary trusses to provide the necessary truss bracing. The center tower crane was removed only after all of the structural steel was connected and prior to the erection of the skylight and stainless steel cladding panels.

The design used high strength slip-critical connections with over-

size and slotted holes.

Other Considerations

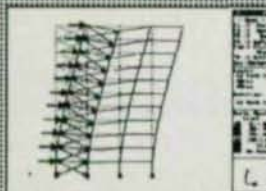
Particular care was taken in evaluating different cladding types. Alternatives considered included anodized aluminum, porcelain enamel, painted steel, and stainless steel. The effects of pollutants, rain, ice, hail, temperature extremes, and window washing equipment were considered. The owner opted for a 2"-thick stainless steel foam panel with metal deck underlayment. The approximately 8,000 panels, each 15' in length, were manufactured by E.G. Smith Construction Products.

Lawrence G. Griffis is senior vice president and director of structural engineering for Walter P. Moore and Associates, Inc., Houston. This article was adapted from a paper he presented at the National Steel Construction Conference in Washington, DC. □

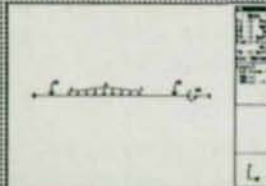
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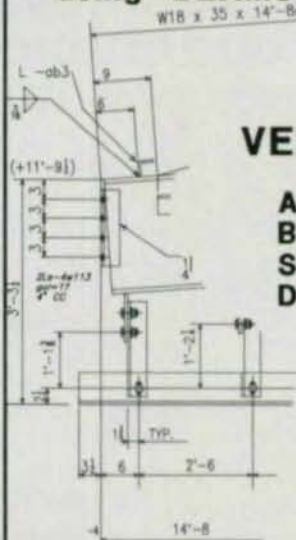
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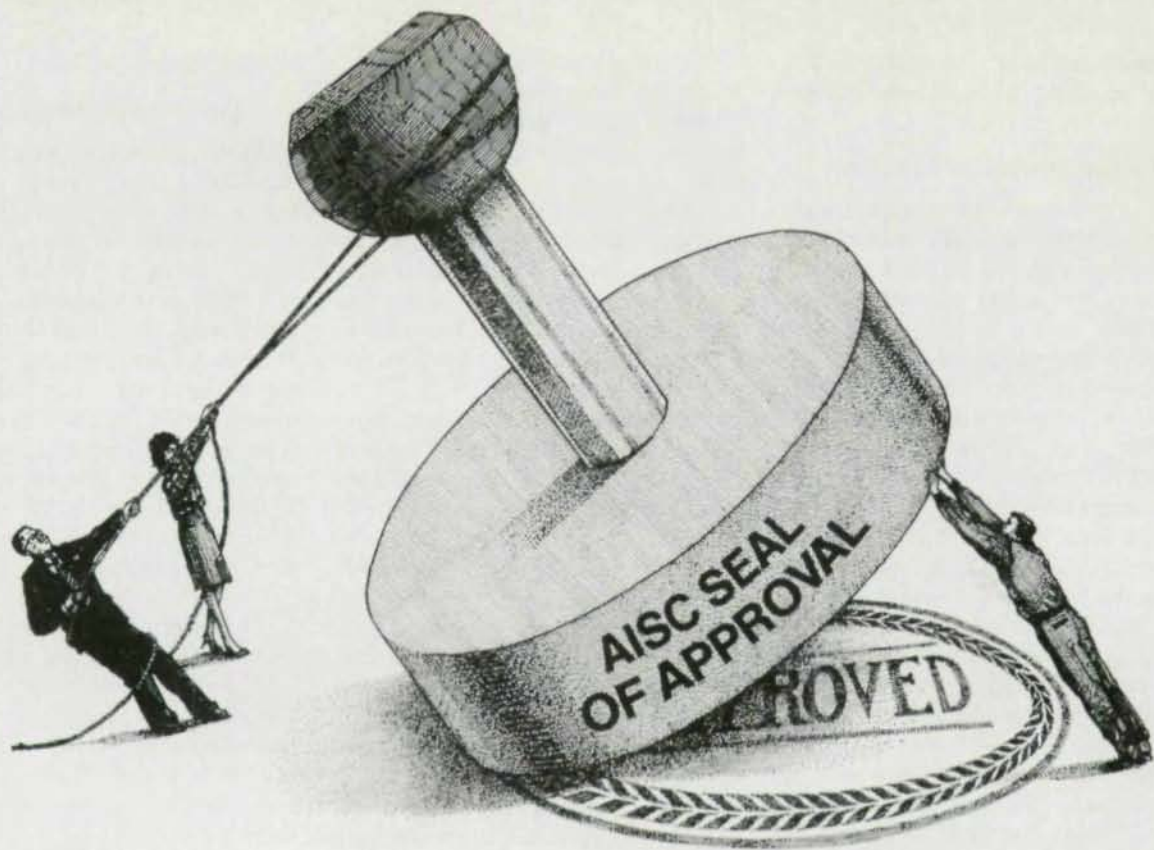
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Vertical Expansion Of Vintage Buildings

An understanding of the design practices employed in the past can greatly simplify a renovation project

By Charles H. Thornton, P.E., Ph.D.; Udom Hungspruke; and Robert P. DeScenza, P.E.

(This is the first part of a two-part article. The second part will be presented in July.)

Like a fine red wine, the value of a vintage building improves with age—not only because of inflation, but also because of the quality of the construction.

We've all heard the adage "they just don't build 'em like they used to." In spite of the deterioration and degradation of structures that occurs when they're not maintained, this adage is still true, especially when applied to existing structural steel buildings.

The quality of these old buildings often means that in many cases significant increased gross and leasable area can be added with little or no strengthening.

Because of the rapid rise in land costs during the past few decades, a keen interest on the part of owners and developers has materialized concerning the vertical expansion of existing buildings. In addition to the cost savings that can be realized by utilizing an existing structure, environmental and zoning approvals are essentially in place. Further, the project can often be completed more quickly than new construction.

But before any plans can be made, it is essential that a structural engineer perform an in-depth

analysis and evaluation. This analysis must include an understanding of the construction methods prevalent when the structure was designed. By reviewing ASTM, AISC and other industry standards, the 20th Century can be divided into six vintage periods: 1900 to 1923; 1923 to 1936; 1936 to 1949; 1949 to 1960; 1960 to 1986; and 1986 to the present.

Five major areas should be explored when commencing an investigation into the potential for vertical expansion.

- Review of as-built drawings (if they exist) and field observations

ments.

- Review of the changes in functional use within the building.

As-Built Drawings

Obtaining complete up-to-date records pertaining to the construction of the building can save much of the time and expense otherwise required for the engineer to probe and measure the actual field conditions.

However, even if drawings are available, it is essential that the engineer confirm that they represent actual conditions, especially in areas that are to be upgraded.

Also, the engineer must carefully inspect the condition of representative structural elements for corrosion, cracks or deterioration.

Once drawing accuracy and structural condition are known, the capacity

of members and connections can be evaluated. Samples of existing steel should be taken as required, and tested to determine physical properties and weldability.

Analysis Methods

Analytical and design methods are continually changing, usually toward more precise methods that reduce the amount of steel required. In general, older structures were designed much more conservatively and have significant excess structural capacity.

Even if drawings are available, it is essential that the engineer confirm that they represent actual conditions, especially in areas that are to be upgraded

and measurements of the existing structure.

- Comparisons of the analysis and design methods in use at the original time of design to present practice.
- Comparison of the requirements of the prevailing codes and standards in effect at the time of the original design to the present requirements.
- Comparison of code provisions for live load and live load reductions at the time of the original design to the present require-

Summary of Common Practice 1900-Present

| | 1900-1923 | 1923-1936 | 1936-1960 | 1960-Present |
|---|--|--|---|--|
| Type of steel | ASTM A7 ASTM A9 Cast Iron | ASTM A7 ASTM A9 | ASTM A7 | ASTM A7 ASTM A36 ASTM A572 |
| Design Method/ State of Practice | No live load red. Working stress Design stress 16 ksi Riveted conn.'s Test steel for weldability | No live load red. Working stress Design stress 18 ksi Riveted conn.'s Weldable steel generally found | Live load reduction Working stress Design stress 20 ksi Riveted conn.'s Bolted conn.'s Weldable steel | Live load reduction Working stress (prevalent) LRFD introduced Design stress 20 - 24 ksi H.S. bolted conn.'s Bolted conn.'s Weldable steel |

1900-1923

Structural design of columns began utilizing Euler's formula for slender columns. By 1910, the double modulus theory of column design became widely used.

Many construction methods and materials were designed and patented by manufacturers and contractors without formal calculations. During this period, the allowable stress in steel members was 16 ksi.

Several catastrophic fires at the end of the 19th Century resulted in major cities enacting building codes to govern construction. These codes were generally empirical and conservative in terms of allowable stresses and imposed loads.

However, due to their arbitrary nature, they may have been "unconservative" in certain areas. The loads given in these codes typically reflected the heavy construction materials, furniture and machinery typical of that period. Live load reductions were not allowed.

1923-1936

In 1923, the first specification of the American Institute of Steel Construction formalized and standardized structural design for the first time. It was based on the use of ASTM A7 and A9 steels, and had an allowable basic working stress of 18 ksi. This standard had a total of five formulae and a set of fixed allowable stresses. During this period, analysis of indeterminate structures was performed using Hardy Cross's technique of moment distribution.

Building codes were very slow to change during this period. Live load reductions began to be incorporated, but they typically were very small and limited to columns. The effect of impact loads was generally acknowledged.

1936-1949

As improved mill practices resulted in more uniform steel, AISC increased the allowable basic working stress to 20 ksi. Research continued regarding inelastic column design and resulted in Shanley's

Theory in 1947 and Johnson's Parabola.

Building codes became more specific and addressed a greater variety of construction materials. Wind loads were understood to vary over the height of structures. Live load reductions were increased.

1949-1960

In 1949, the *Manual of Steel Construction* adopted the tangent modulus theory for the design of short columns. This edition also shifted from an allowable basic working stress with reductions, to calculations based on steel yield strength with safety factors. Research laboratories such as the one at Lehigh University produced technical data for analysis and development. It was at Lehigh that residual steel stresses were discovered during the 1950s.

The codes continued to become more specific. The allowable stress increases associated with wind loads were better defined. Snow loads were revised to better address

drift conditions. Building codes continued to increase the variety of construction materials addressed. Most codes continued to provide formulae and design guidelines for the design of structural members, although standards such as those in the *Manual of Steel Construction* were gaining increased acceptance.

1960-1986

Analysis of complex structures was made vastly easier in the 1960s with the entrance of computer capabilities into design offices. Designers took advantage of these new tools and new design methods such as composite beam design to make designs more efficient. The *Manual of Steel Construction* was continuously updated to incorporate new steel types and in 1969, it revised the column design formulae to address residual steel stresses.

Live load requirements were adjusted to reflect more modern usages and occupancies. Wind load requirements were revised to incorporate intensive laboratory research by the American National Standards Institute.

1986-Present

Design methods continue to improve. The new *AISC Load and Resistance Factor Design Manual* makes it possible to design steel based on its ultimate strength and permits greater efficiency in design. Computer techniques continue to improve with optimization programs to make designs more efficient. Desktop computers make complex analysis techniques available to even the smallest office. Seismic and wind requirements also continue to be reviewed and enhanced.

Charles H. Thornton is chairman and principal of the New York City-based consulting firm of Thornton Tomasetti Engineers. Udom Hungspruke is a vice president and Robert P. DeScenza is an associate with Thornton Tomasetti Engineers. This article was adapted from a paper presented at the National Steel Construction Conference. □

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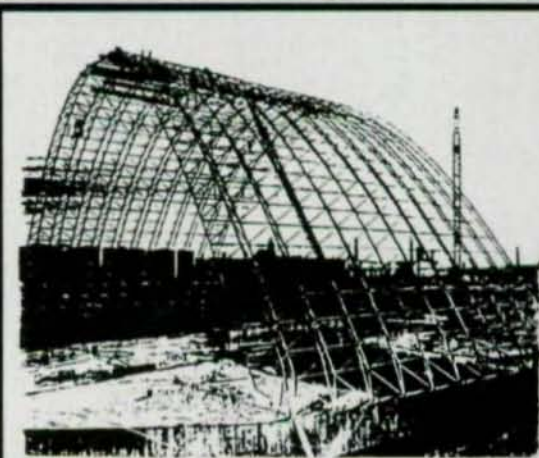
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Vertical Expansion To Add 235,000 Sq. Ft.

By Charles H. Thornton, P.E., Ph.D.; Udom Hungspruke; and Robert P. DeScenza, P.E.



The proposed addition to the building would add one story to the courtyard area, four stories to the center portion, and seven stories to the new "tower" section.

When it was built in various stages between 1906 and 1913, the B. Altman Department Store was the architectural centerpiece of New York's commercial district. Today, it's a registered historic landmark occupying an entire city block from Fifth Avenue to Madison Avenue and from 34th Street to 35th Street.

Most of the building is nine-stories high, except for the 120'-wide portion on the east side, which is

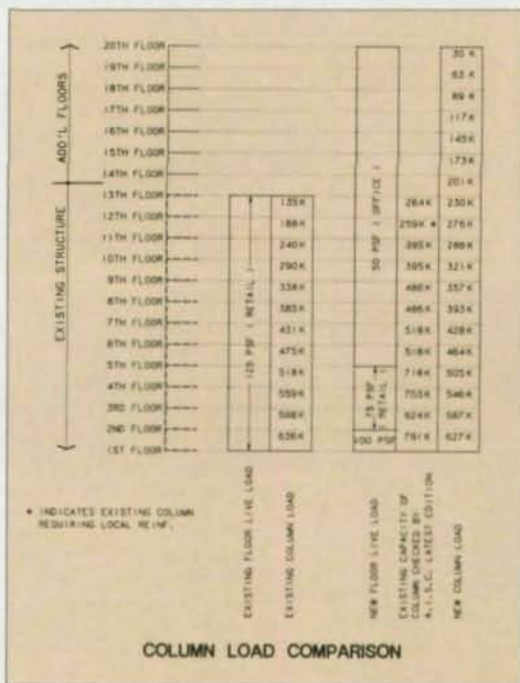
What makes the project possible is the trade-off of loads, which required the lightest possible floor system to work

13-stories high and is known as the Bustle Area.

The proposed vertical expansion approved by the Landmark Commission will make a drastic change in the appearance of the building. The expansion consists of adding height to the building in three zones:

- Courtyard Area. One additional story on the west side of the building.
- Pavilion Area. Four-stories in the center portion of the building.
- Tower Area. Seven-stories over the entire 120'-wide Bustle area of the structure.

The three additions will raise the building to a total of 20 stories and will add 235,000 sq. ft. to the



existing 900,000 sq. ft.

When the project was conceived, there was a desire to add as many floors as possible to the Bustle area. A major constraint was that any required reinforcement of the columns could only be provided above the sixth floor slab. Any construction below the sixth floor slab was not allowed because of its impact on the existing retail activity, which would remain in operation up through the fifth floor of the building.

After careful evaluation of the reserve capacity in the existing columns, it was found to be feasible to add seven floors. In addition to using the lightest possible floor framing system, the addition was made possible by various changes in design and construction methods.

Design Methods

The B. Altman building was built in the early 1900s when the prevailing allowable stress in structural steel members was 16 ksi. At present, the allowable stress is .66 fy. For ASTM A7 steel with today's code, the allowable stress is equal to $.66 \times 33 \text{ ksi} = 22 \text{ ksi}$. Using this item, the increase in total capacity of the existing columns is approximately 35%.

Loading Requirements

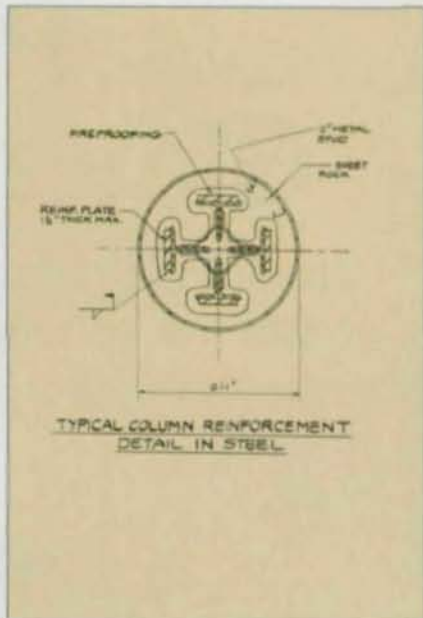
The occupancy loading requirements have gone through many changes during the past 85 years. In the case of the B. Altman building, the entire building was designed for retail use, which, at that time, required a 125 psf live load. The current New York City code requires a 75 psf live load for retail areas at grade. Using this item, 50 psf in reserve capacity is realized. With a typical column spacing of 21' x 21', this reserve capacity amounts to approximately 22 kips per floor.

Occupancy Changes

After it is renovated (the project is currently on hold as a result of



Many of the existing steel columns were strengthened with reinforcing plates on the flanges of the columns.



New York's glutted office market), the B. Altman building will have a mixed-use occupancy consisting of retail through the fifth floor and office space above.

The current New York City code requires 50 psf live load for office areas. Therefore, 75 psf in reserve capacity is realized in those areas where the office occupancy replaces retail area.

Live Load Reduction

In the office area where the live load is 50 psf, the current New York City code allows a reduction in live load based upon the tributary area supported by the element being designed. For columns and foundations, the live load reduction could be up to 60%.

Roof Requirements

The existing roof above the 13th floor was originally constructed with slopes to drain. A thick cinder fill with an approximate average weight of 140 psf was used to create the slope. Since the slope rendered the roof unusable as a floor for the addition, it was replaced with a lighter concrete slab on

metal deck with an approximate weight of 45 psf. This replacement of floor structure contributes to additional reserve capacity.

Two schemes were devised for those columns that needed additional reinforcement: either providing reinforcing plates on the flanges of the existing columns or encasing the existing column with concrete. Adding reinforcing plates is simpler and results in a smaller overall section, though the built-up column requires fireproofing and finish column encasement.

In both schemes, proper transfer of the load in the reinforced columns must be achieved at each of the floor slabs.

What makes the project possible is the trade-off of loads, which required the lightest possible floor system to work.

The system chosen was a steel frame with a composite slab of 2" metal decking and 2 1/2" of lightweight concrete. The new W12 columns are located directly above each of the existing columns so loads are evenly distributed. The structural framing is designed using W12 beams and W16 girders. □

Preserving Cleveland's History

The interior of this landmark was completely rebuilt above the second floor to create modern office space



The interior of this 1888 building was completely rebuilt to create modern office space. Photography by Mort Tucker Photography.

By Gary E. Thayer, P.E., and Kurt K. Rim, P.E.

Cleveland's resurgence during the last decade unfortunately resulted in the demolition of some of the city's historical past. But with the growth of the preservation movement, important structures are increasingly being saved.

The Society National Bank Building, located on the north side of Public Square in downtown Cleveland, is a 12-story red sandstone office tower designed by Burnham and Root in 1888. In its day, the National Register building was the tallest "Skyscraper" between New York and Chicago.

The building's main feature is its ornate Arts and Crafts two-story banking hall with coffered, luminous, stained-glass ceiling.

At the south end of this banking hall is a mezzanine level that is entirely devoted to use as a board room for Society Bank. The board room is a rich space, finished on all surfaces with hand carved oak, and would be impossible to economically duplicate today.

The interior of the building is constructed of a steel post and beam skeleton with 15" of flat clay tile arch construction originally overlaid with a floor system composed of cinders and wood sleepers, then finished with oak flooring.

At the core of the steel-framed building was an atrium light well capped at the roof level with a delicate steel-and-glass skylight that, over the years, was covered with insulation and roofing and rendered unusable.

In 1987, The Osborn Engineering

Co. was retained by the building's owner, The Jacobs Brothers Co., Cleveland, to plan a complete renovation of the building. The knowledge and insight of the owner's representative, Thomas Henneberry, was crucial to the project's success. The renovation was part of a larger project that included the construction of a new office tower next door.

Initial Investigation

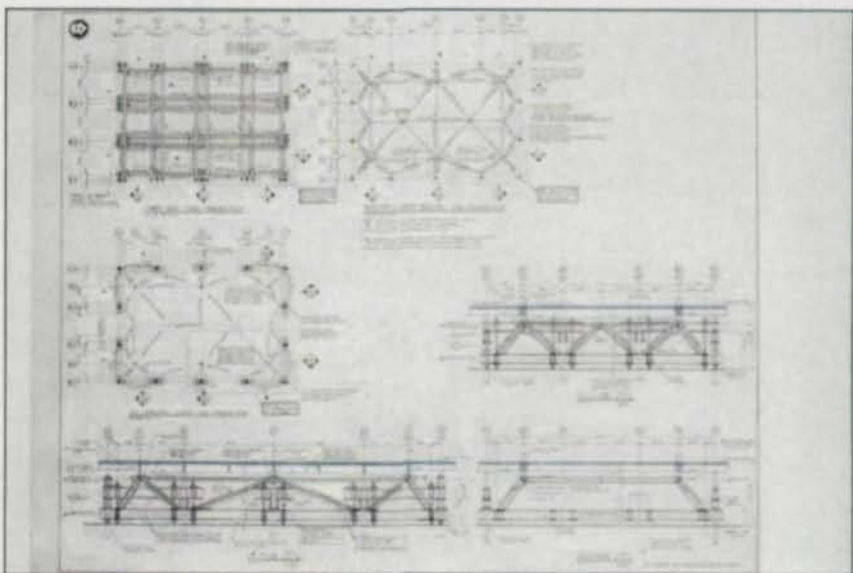
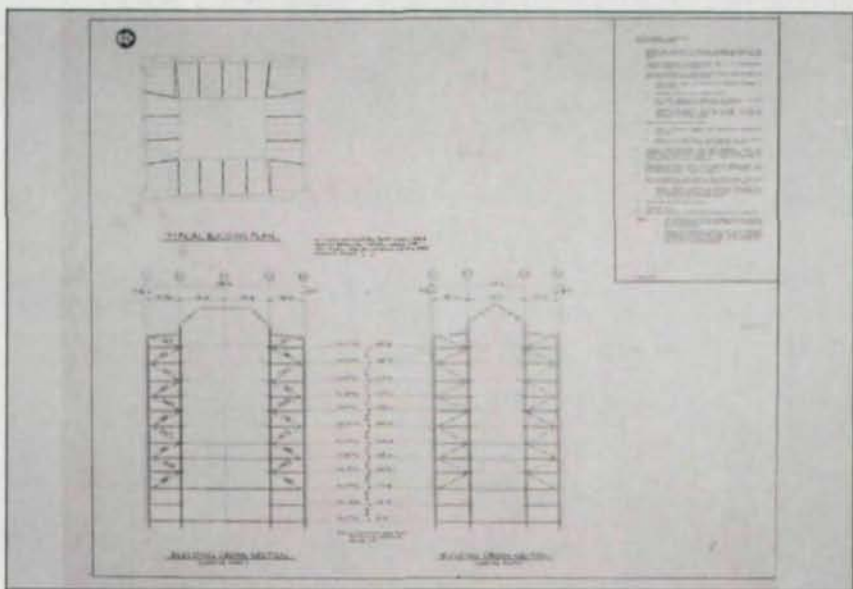
The initial investigation concentrated on three areas. Was the existing floor framing capable of supporting Class A office loading requirements? Could the existing atrium space be converted into Class A floor space without overloading the existing foundation? And could the existing building be tied into the new tower?

Structural analysis based on existing documentation revealed the existing floor framing to be inadequate to support the heavy dead load of the existing tile arch floor and also satisfy the live load requirements for office space under today's standards.

The existing floor loads totaled 126 psf, with the bulk coming from 4½" of Haydite Concrete Fill (41 psf) and the flat tile arch floor (68 psf). Since the existing floor was capable of supporting total loads of 150 psf, the live load capacity was only 24 psf, which was clearly inadequate. However, structural reinforcement was not practical without removing the tile arch floor.

With the tile arch floor removed and replaced with a cellular floor deck, dead load would be reduced to 44 psf, leaving in excess of 100 psf for live loads.

The major disadvantage to this scheme, though, would be that floor heights would not be uniform to match the adjacent tower, which the owner planned to connect to the old building with pedestrian bridges. Also, there would be problems in retrofitting HVAC and lighting to the extent that 9' clear ceilings could not be





The superstructure supports the six new core columns and transfers their reactions to the lower grid.

maintained.

After analyzing the existing conditions, the engineer proposed a bold plan to structurally renovate the entire building from the second level on up. The existing facade would be restored, the banking hall saved, and the interior floors above the second level realigned to match the tower. Also, the existing atrium would be filled in at each level to increase rentable space. The plan also called for reducing the number of interior core columns from 14 to six and aligning them with the space planning module of the tower.

In order to succeed, the design required a huge transfer structure located above the banking hall and occupying the entire second floor. Initial estimates were for a structure weighing some 250 tons.

Structural Analysis

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structural field survey to define the capabilities and limitations of the 100-year-old structure. Included were tests to verify the composition and weldability of the steel skeleton, as well as verify locations of critical columns and beams and identify the types of connections and foundations.

One of the first areas explored were the footings. Excavations were made in the basement floor adjacent to the stepped sandstone footings by the contractor, Turner Construction Co. It was discovered that the large sandstone foundation was laid up from a grillage of rails poured solid with concrete and it had an allowable bearing capacity in excess of 5,000 psf.

At the same time that the foundations were being investigated, basic column sizes and locations were being verified at the upper levels. Center-to-center dimensions were thoroughly checked against

references and walls and pilasters were opened on every floor to check orientation and cross section.

The thoroughness of this investigation proved crucial. Not only were the existing columns oriented differently from the references, they were not of the same cross section or material as referenced. As we would learn throughout this project, when renovating a structure, make no assumptions. Verify everything. In this case, columns that were referenced as circular and made from cast iron were actually built-up of steel "Z" sections and plate.

Materials Testing

When coupons were taken from the existing structure, another surprising discovery was made. The steel used on this building 100 years ago was the near equivalent to today's A36 in both chemistry and strength.



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However, a potential problem was noted in terms of weldability. An elevated "spike" in the phosphorous content of the low carbon alloy meant the potential for cold cracking of welds used to make connections. Fortunately, visual and bend tests made by a testing laboratory in accordance with the Structural Welding Code proved that the material could be welded

without a problem.

Transfer Structure

By replacing the old, heavy floor system with a modern, light-weight system, it was possible to rebuild the entire interior of the structure above the second floor—including infilling eight floors of atrium space—without reinforcing the existing columns. The rebuilt floors increased the leasable area of the

building by 30%.

The 14 existing columns were left in place up through the second floor. Attached to the existing columns at the second level is a transfer structure, which in turn supports six new columns.

The transfer structure had three distinct components: superstructure; lower grid; and hanger support steel.

The superstructure supports the six new core columns and transfers their reactions to the lower grid. It includes "knuckles", which connect the columns and the primary load transfer members. These members include one "vertical" and two "diagonals" at each knuckle.

The lower grid is a sturdy and highly redundant steel grid suspended by rods approximately 1' above the existing second floor. It acts as the lower portion of the three-dimensional, space frame-type transfer structure and takes the resultant tension forces. Also, the lower grid acted as a work platform during construction keeping the heavy loads of the superstructure off of the second floor.

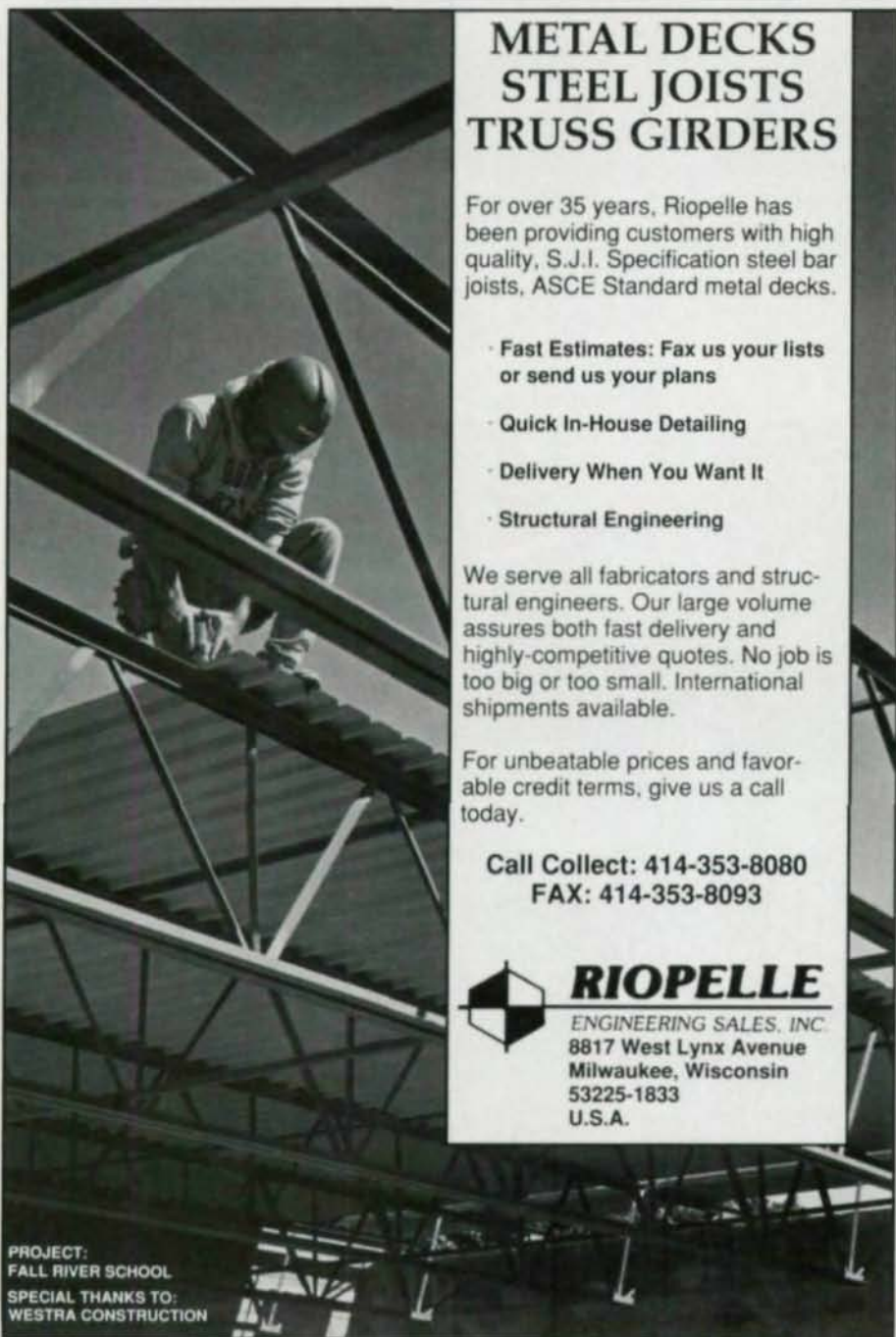
The hanger support steel includes 28 6"-diameter steel "hanger rods" that suspend the lower grid and the hanger support beams and plates that transfer vertical reactions from the lower grid into the 14 existing interior columns at the second level.

Structural Design

The new structural system was analyzed using the Ranul Stress computer software package from Ranul, Inc., Baltimore.

The basic analytical problem was how to direct the reaction of the six new columns in the proper proportion through the transfer structure.

The solution was for the diagonals in the transfer structure to be selected and detailed to encourage the load from the new column to distribute down the diagonal proportionally to adjacent columns rather than vertically to the nearest



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column.

By holding the geometry constant, the controlling factor affecting the design was the proportional areas of the members. When using a stiffness-based program for design of a structure, usually the final solution is obtained by manipulation of relative member stiffness—that is, the moment of inertia divided by length of the member. In this case, however, even the smallest members were extremely stiff and manipulation of member forces was found instead to be a direct function of their area. Members were then chosen based on ratios of their areas and the optimum reactions in the hangers were kept within allowable limits.

A safety factor of 2 against ultimate failure was chosen to govern allowable stresses in the transfer system. Anything higher resulted in members of extreme size, larger than commonly rolled.

The six major joints or "knuckles" in the transfer system were detailed to direct the load into the diagonal and thereby control overload to the nearest column. This was accomplished by using full penetration welded moment connections of the bent composed of the two diagonals and the main top chord compression element in the connection. A relatively thin flexible plate type, bolted, bearing connection was then used to simulate a "pin" type, moment free condition at the top of the vertical member of the joint.

Structural Components

To reinforce the lower grid, 3"-thick bearing plates were used where reactions were transferred to the hanger rods. Large tension and bending forces required design of the lower grid with double and, in some cases, triple beams. These were welded together in the shop and preassembled in 10 pieces for shipment by trailer to the site.

Each hanger for the transfer structure has a working load of 350 kips. As a result, 6"-diameter Grade 42 steel rods were designed. Each



The joints in the transfer system were designed to direct the load into the diagonal and thereby control overload to the nearest column.

end of the rods was threaded to allow for adjustment necessary to level the transfer structure and "tune" the proportional load to that particular hanger.

Hanger rods were held in place by a system of hanger support beams. These were made of 24", Grade 50 steel beams attached to columns with a combination of 3/4" high strength A490, Type F bolts and welded with E7018 low hydrogen electrodes.

Existing 3/4" rivets were removed from the existing columns in the area of hanger steel attachment. These holes were then either used for bolting or they were ground smooth and plugged and then new holes were drilled.

Reactions were high enough that additional W24 "seats" were added to develop the required safety factor for the connections. Hanger support beams at corners of the column grid were combined with those of adjacent columns to account for local eccentricity at corner columns. The local eccentricity of one corner column was balanced globally by the opposite diagonal corner column, thus

maintaining stability of the system as a whole.

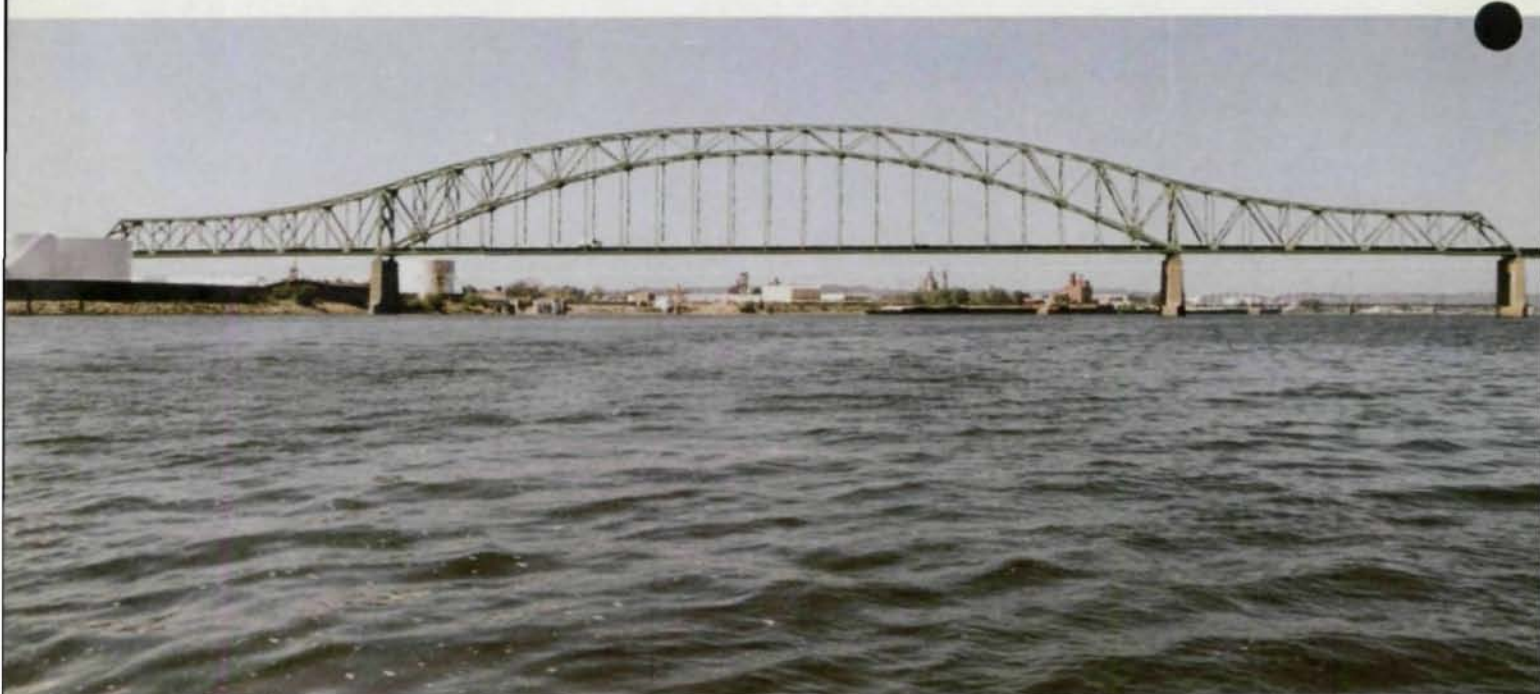
Floor Design

Floor design for the renovated building was of critical concern.

Grade 50 steel was used to reduce the depth of members to allow more room for lighting and HVAC ductwork. Blended cellular composite floor deck was used throughout with Nelson studs for composite beams. The floors had an overall thickness of 5 1/2" and used regular weight concrete fill.

While the floor system was initially designed using Allowable Stress Design, it was redesigned after the engineers attended an ASIC seminar on Load and Resistance Factor Design. The redesign created a weight savings of 12% over the original design.

Gary E. Thayer is a senior project manager and Kurt K. Rim is executive vice president and director of civil/structural engineering for the Osborn Engineering Co., Cleveland. This article is adapted from a paper presented at the National Steel Construction Conference in Washington, DC. □



Bridge Expansion

A beautiful steel bridge was economically strengthened and modified to handle heavier loads on a wider and safer roadway

By Arthur W. Hedgren, Ph.D., P.E.
and William Schmitt, P.E.

Spanning the Mississippi River between Dubuque, IA, and East Dubuque, IL, is the 48-year-old Julien Dubuque Bridge. Due to deteriorated conditions, the bridge requires rehabilitation. But at the same time, since traffic on the bridge is expected to increase from a 19,000 vehicles on an average day to 21,900 in the year 2008, the Iowa DOT decided to also expand it.

The project included field inspection, widening the deck, member strengthening, and structural repairs. The Pittsburgh office of HDR Engineering, Inc., was the engineer.

The total length of the bridge between centerlines of abutment bearings is approximately 5,760', including 1,448' of girder approach spans on the Iowa side, a three-span through arched truss unit 1,539' long over the river, and 2,773' of girder spans on the Illinois approach.

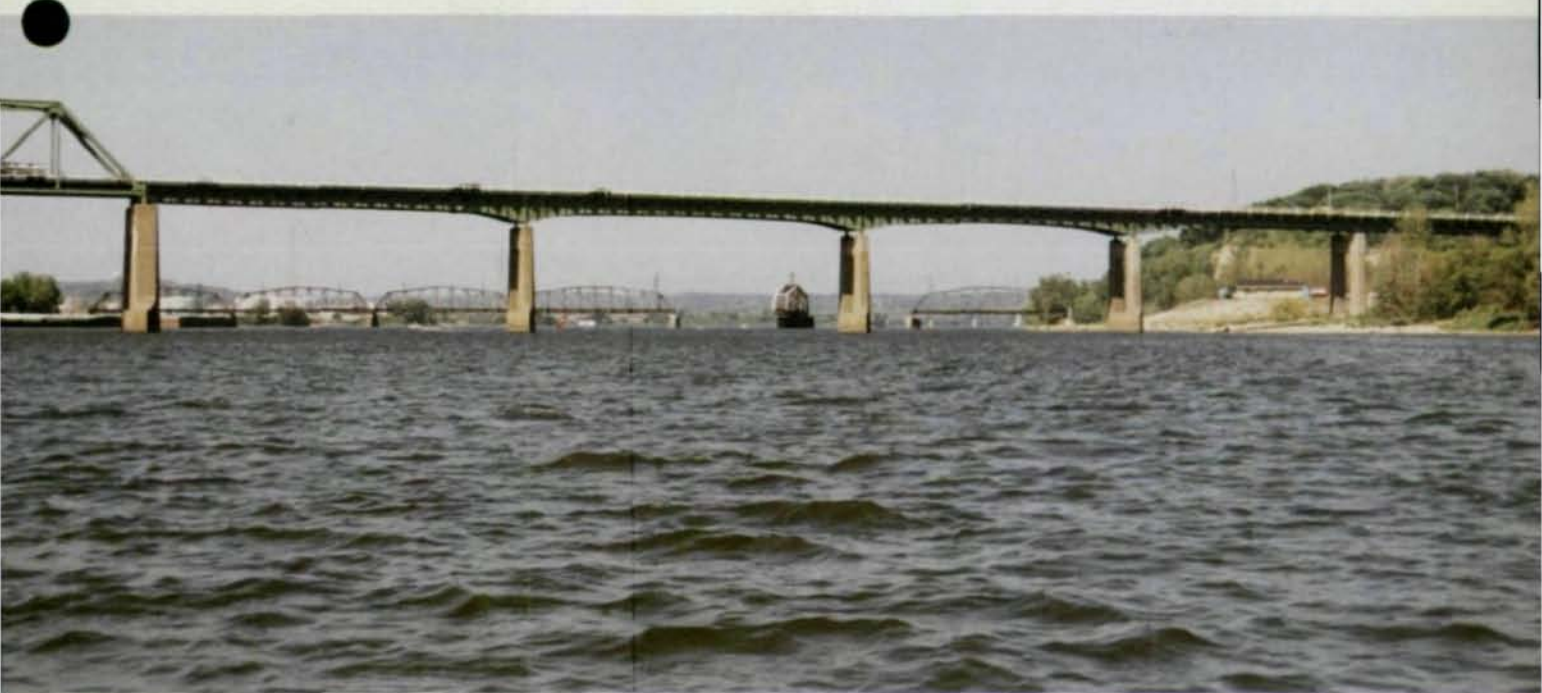
The existing bridge deck pro-

vides a 24' roadway curb-to-curb, a 5' concrete pedestrian sidewalk on the north side, and a steel safety walk on the south side. The remodeled curb-to-curb width will be 28' with 4' widening on the north side. A 4' 6" sidewalk will be located on the south side of the remodeled structure.

Bridge Renovation

The westernmost 512' of the Iowa approach spans is being entirely reconstructed. This reconstruction extends from the existing west abutment to the hinge point in the existing Span 10. The renovation will remove the roadway deck, sidewalk, safety walk, and railings and the remaining 5,248' length of the bridge—from the hinge point in existing Span 10 to the existing east abutment—will be remodeled.

The deck replacement and rating analysis of the steel structure for the alternatives included: cast-in-place (both normal weight and



Handles Additional Traffic

lightweight concrete); precast (both normal weight and lightweight concrete); orthotropic (steel orthotropic); exodermic (steel grid and reinforced concrete composite of either normal weight or lightweight concrete); and a partially-filled steel grid (both normal weight and lightweight concrete).

Design Criteria

The Service Load Design Method was used in the analysis of main truss members. The Strength Design Method (Load Factor) was used for girder spans and the floor system of the truss spans.

Carbon steel ASTM A7 or silicon steel ASTM A94 were used for the existing structural components with yield strengths of 33,000 psi and 45,000 psi, respectively. Ten silicon steel test samples were taken at various truss member locations to determine if a yield strength of 50,000 psi could be used for the silicon steel. However, test results indicated yields be-

tween 45,000 and 50,000 psi. Therefore, the analysis for strengthening the silicon steel members is based on 45,000 psi.

Design live loading was increased from H20, used in the original design, to HS20.

Truss And Approach Spans

Due to the Iowa DOT's preference, regular weight concrete was used despite requiring a larger number of members to be strengthened and considerably thicker strengthening plates.

Girders in the approach spans were strengthened by adding and/or increasing the lengths of top flange cover plates and by providing shear connectors to obtain a composite designed section. Also, struts were added to brace bottom flanges adjacent to piers.

Upgrading the live load from H20 to HS20 required strengthening of all floor beams in the truss spans and in the girder spans. Floor beams in the truss spans

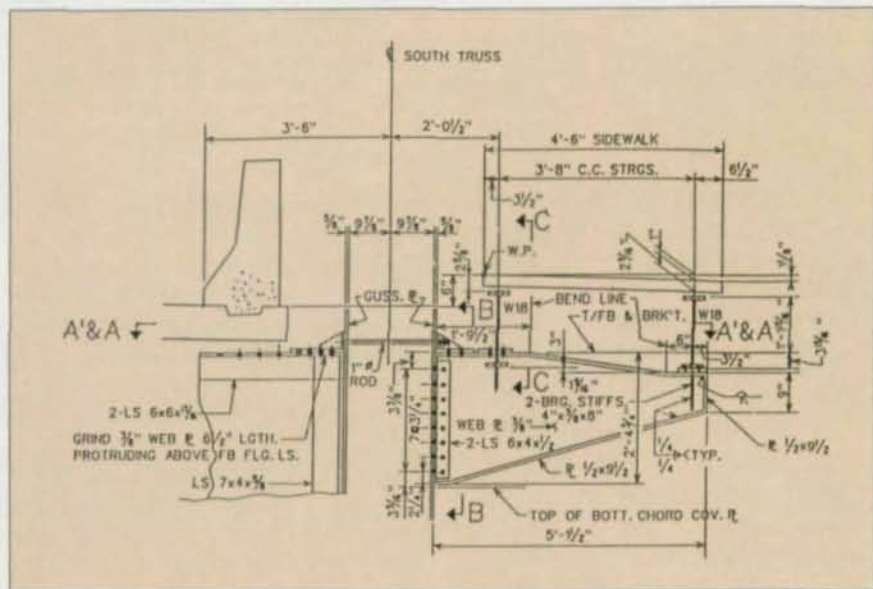
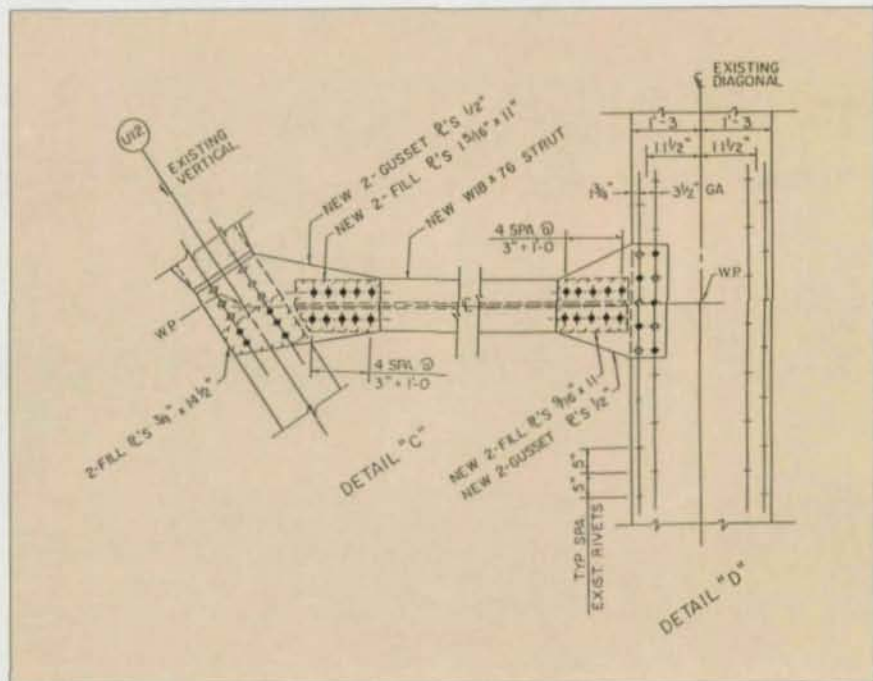
were strengthened by adding shear connectors, while floor beams in the approach spans were strengthened by adding only a bottom flange cover plate since the stringers are continuous over the floor beams.

Because the deck beams on truss spans prevented adding plates to the top of stringer top flanges, the existing south stringer was strengthened by adding web side plates near the top of the stringer and a bottom flange plate. The addition of a new stringer on the north side eliminated the need to strengthen the existing north stringer.

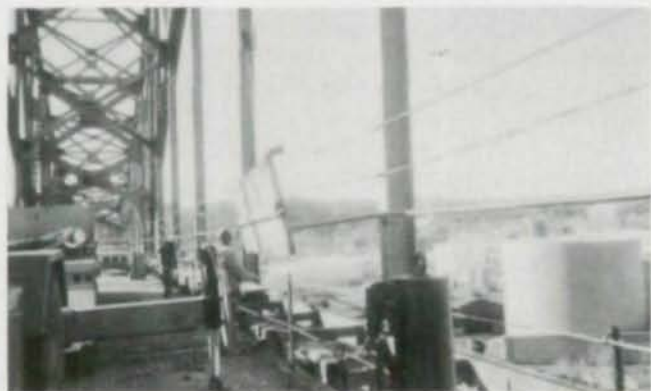
Additional Stringer

Widening of the roadway in the truss spans necessitated adding an additional stringer on the north side and extending the deck beams. Also, sidewalk brackets are attached to the outside of the south truss.

Widening of the approach spans



Shown at right is the sidewalk removal on the existing portion of the Julien Dubuque Bridge over the Mississippi River.



required replacement of the sidewalk stringer and replacement of all brackets. Moving the sidewalk to the south side required replacement of all the existing curb brackets with sidewalk brackets and the addition of a new sidewalk stringer.

Roadway Widening

The roadway widening and relocation of the sidewalk necessitated modification of the East Abutment and the approach roadway. The approach roadway plans were developed under subcontract to Lee & Batheja, Omaha, while HDR designs included the development of sidewalk details and wall brackets to carry the sidewalk around the existing south pylon. The design of the bridge deck, railing and fence over the bridge was subcontracted to Greiner, Inc., Timonium, MD.

The rehabilitation of the bridge included replacement of all deck expansion joints with sealed types of joints. Deteriorated steel components at the joints were repaired or replaced as required.

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A low bid of \$13,407,514 was submitted by Johnson Bros. Corp. of Litchfield, MN. The bridge was closed to traffic on Feb. 4, 1991, and is to be reopened on Dec. 1, 1991.

Arthur W. Hedgren, Jr., is a vice president and William Schmitt is a consultant with the Pittsburgh office of HDR Engineering, Inc., a well-known engineering firm headquartered in Omaha. This article is adapted from a paper they presented at this year's National Steel Construction Conference in Washington, DC. □

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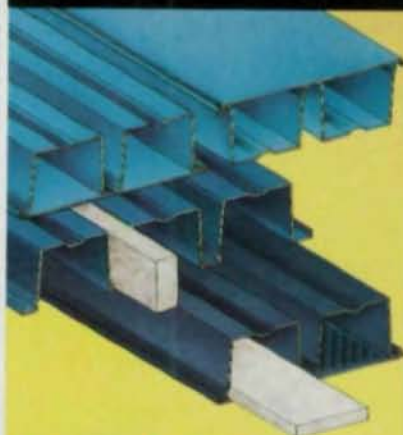
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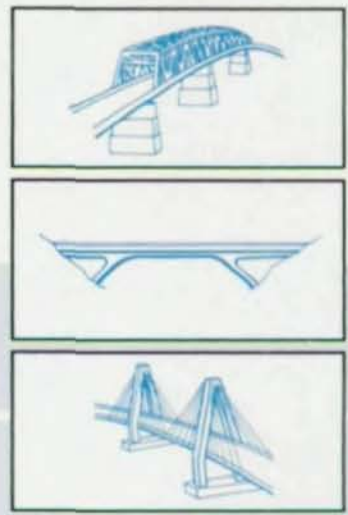
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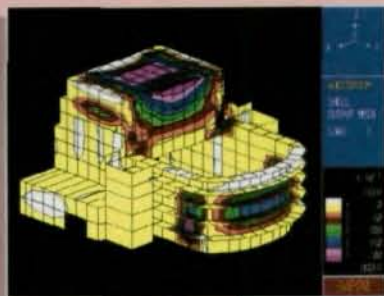


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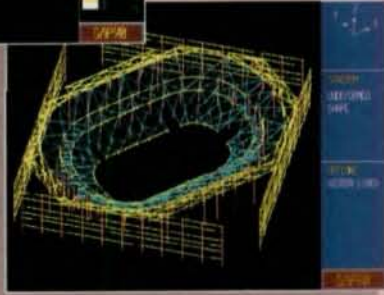


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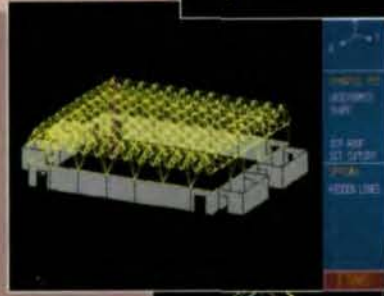
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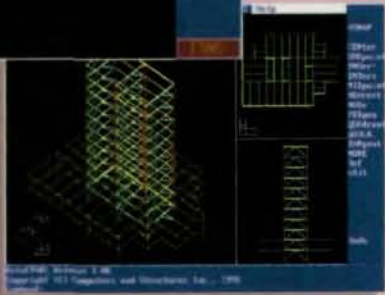


Complex Frame Analysis

Space Frame Design



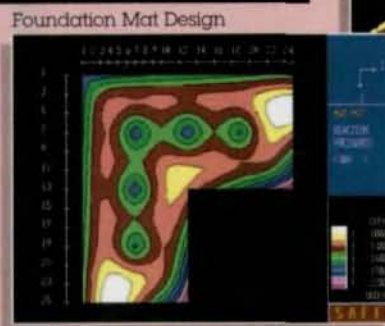
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