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April 1994

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After Olive View Hospital collapsed in the 1971 San Fernando Valley Earthquake, it was re-built in steel. Despite a 2.3 g acceleration at the rooftop level during the 1994 Northridge Earthquake, the only damage was to some mechanical and sprinkler equipment.

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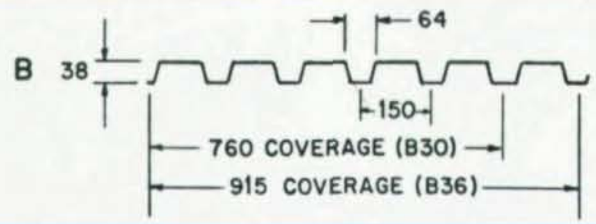

GAGE	THICKNESS mm	I mm^4/mm	S_p mm^3/mm	S_n mm^3/mm
22	0.749	232	10.2	10.7
20	0.909	301	13.4	14.0
18	1.204	423	18.3	19.4
16	1.519	546	23.7	24.[illegible]

TYPE B, BI, BA, BIA

Uniform Total Load (Dead + Live), kPa

Span Type	Gage	Span, mm 1500	1650	1800	1950	2100	2250	2400	2550	2700	2850
Single	22	*5.0*	*3.8*	*3.1*	*2.5*						
	20	*6.3*	*4.8*	*3.8*	*3.1*	*2.6*	*2.2*				
	18	*8.7*	*6.6*	*5.2*	*4.2*	*3.5*	*2.9*	*2.1*	*1.9*		
	16	*11.0*	*8.4*	*6.6*	*5.3*	*4.3*	*3.6*	*3.1*	*2.6*	*2.3*	*2.0*
Double	22	5.2	4.3	3.6	3.1	2.7					
	20	6.7	5.6	4.7	4.0	3.5	3.0	2.7	2.4		
	18	9.3	7.7	6.5	5.5	4.8	4.2	3.7	3.3	2.9	2.6
	16	11.6	9.6	8.1	6.9	6.0	5.2	4.6	4.1	3.6	3.3
Triple	22	6.4	5.3	4.5	3.8	3.3	2.9				
	20	8.3	6.9	5.8	5.0	4.3	*3.7*	*3.1*	*2.7*		
	18	11.5	9.6	8.1	6.9	6.0	5.0	*4.2*	*3.6*	*3.1*	*2.7*
	16	14.4	12.0	10.1	8.6	7.5	6.4	*5.3*	*4.5*	*3.9*	*3.4*

Loads shown in *italics* are controlled by L/240 deflection. Dead load is assumed to be 0.48 kPa.

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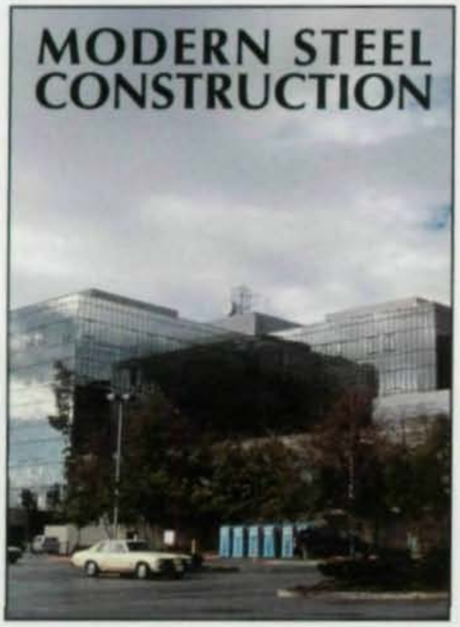
MODERN STEEL CONSTRUCTION

Volume 34, Number 4 — April 1994



After Olive View Hospital collapsed during a 1971 earthquake, the administrators vowed that the rebuilt structure would withstand the next seismic event—and the new steel hospital came through the Northridge Earthquake structurally intact. Coverage of the earthquake begins on page 16.

FEATURES

DEPARTMENTS

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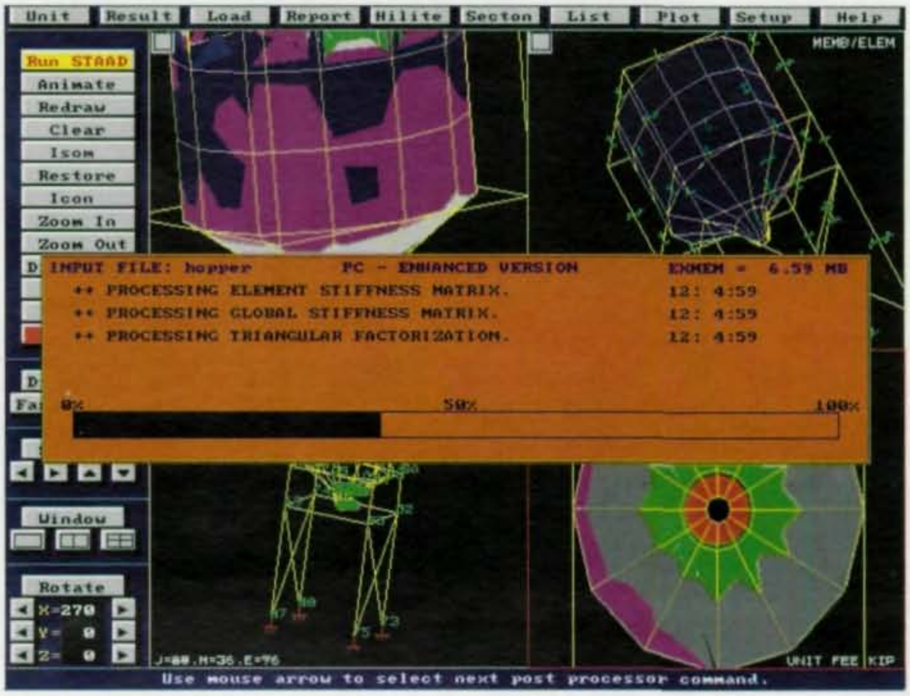

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Steel Performance

While the Northridge Earthquake exposed some chinks in the once-impervious reputation of steel buildings in seismic zones, it is important to remember that from a life-safety standpoint, the performance of steel-framed structures was without peer. Current U.S. building codes are intended to ensure that buildings do not collapse—and in that regard steel-framed buildings came through with flying colors. As one engineer put it, he'd rather be standing in an old steel building during an earthquake than a new building of any other material. Our report on the earthquake design begins on page 16.

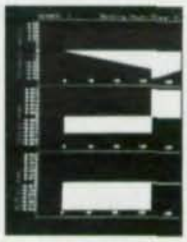
Some steel structures exhibited localized weld failures and flange cracking. The welds have already been repaired in this example.

However, there were some valid complaints about the performance of some steel-framed structures. The first to appear in the popular media dealt with ancillary building damage—that is, damage to lighting fixtures, mechanical equipment, partitions, exterior cladding, etc. While this type of damage also occurred in concrete structures, it was more noticeable in steel structures because it was the only visible damage in steel-framed buildings. Unfortunately, this problem is related to current building code requirements, which emphasize life safety and all but ignore building performance. Most ancillary damage can be prevented; however, unless required by code many engineers and developers are willing to gamble on the infrequency of seismic events (and the availability of adequate insurance) and are unwilling to spend the extra money up-front to prevent these problems.

The second problem was more serious, though again, it was not a life-safety issue. Connection failures occurred in a small number of low- to mid-rise steel-framed structures with moment connections. Typically, these localized failures were realized as weld fractures. In some cases, there was flange cracking, usually located immediately above the top flange of the beam. AISC has already assembled a task group of members of the AISC Seismic Design Committee to study the problem and to propose simple-to-implement solutions. Renovation work is underway on all of the effected structures, and in most cases, will have already been completed by the time you read this. (For more information on these problems, see the article beginning on page 22.)

Note, though, that while the steel industry considers the weld failures and flange cracking to be serious problems, in no case did they lead to the collapse of a structure. From a life-safety standpoint, steel-framed buildings performed flawlessly. **SM**

How to get from here



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Steel Interchange

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

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

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

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

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

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

When was the Vierendeel truss first utilized, why was it named, and for what contributions to structural engineering was he/she recognized?

The Vierendeel truss appears to have been developed in the early 1800's but was not commonly known until early in this century. During the 1800's, there was wide experimentation in the design of bridges, mostly for railroad expansion. Engineers of the day developed new structural configurations and used relatively new materials (such as cast iron) in their designs in order to increase spans and improve structural safety and economy. The first use of what is known today as a Vierendeel truss appears to have been in the cast-iron bowstring design of the Bergues Bridge proposed in 1829 by Guillaume Henri Dufour, the French engineer. The design called for a cast-iron, plate-girder arch with a timber deck suspended from the arch. The characteristic Vierendeel geometry was achieved by providing rectangular openings in the web of the arch sections as they were cast. This concept appears to have evolved from the previously successful use of block-shaped iron cages called voussoirs (after their masonry counterparts) in arched bridges. Later, the pierced-plate design was used for a bridge in Ghent by two Belgians named Marcellis and Duval in about 1844. Arthur Vierendeel, also a Belgian, popularized the form at the start of this century. Today, the term Vierendeel truss has lost its historical origin and is used to describe a specific structural geometry without regard for materials selection and construction method. A similar generalization has occurred with other common truss configurations attributed to Fink, Howe, Pratt, and Warren. Additional information regarding the work of Vierendeel can be found in the following references:

Elton, J. (1982), *Bridges, Docks and Harbours with Related Works*, London, Catalogue 45, B. Weinreb Architectural Books Ltd.

Peters, T. F. (1987), *Transitions in Engineering*, Boston, Birkhauser Verlag.

Vierendeel, A. (1903), *La Construction architectural en fonte*, fer et acier, Louvain.

Richard J. Schmidt
University of Wyoming
Laramie, WY

When asked to design a temporary bracing system for steel beams and columns during the erection phase of construction, what loads are used and what factors of safety are employed for the bracing and its connections?

A 96-member committee of ASCE, under the writer's chairmanship, has been developing the ASCE Guide/Standard for Design Loads on Structures During Construction. Along with dead and live loads, the document deals with environmental loads at short-term exposures and construction loads due to various activities. It specifies maximums as well as point-of-time values of construction loads in various combinations. It is the first ever comprehensive document to specify design loads, load factors and load combinations for structures during their construction phases and for temporary structures in construction. A preliminary working draft was issued for comments in February, 1993. The document is expected to be ready for balloting by the ASCE standards committee later this year, and issued as an ASCE Guide or Standard in 1995.

Robert T. Ratay, PhD, PE
Manhasset, NY

Steel Interchange

When designing using the ASD manual, what is the allowable weak axis bending stress on channel?

In the AISC, Manual of Steel Construction, ASD, 9th Edition, the basic allowable bending stress on any laterally stable or adequately braced member is $F_b = 0.6(Q)F_y$ where "Q" is a local buckling reduction factor given in Appendix B. This is true for both major and minor axis bending. AISC classifies sections into three basic categories. "Compact", "Non-compact" and "Slender-Element" (Section B5). The bending allowable depends on which of the three categories the section falls into, as well as the lateral stability of the section. The slenderness of the individual elements that comprise the shape, as measured by width to thickness ratios, determines into which of the three categories the shape falls, (Section B5, Table B5.1). Broadly speaking the three categories may be thought of as follows:

"Compact sections" are those in which the section's elements are proportioned such that the full plastic moment, $M_p = F_y(Z_x)$, may be reached prior to local buckling.

"Non-Compact sections" are those sections whose elements are proportioned such that the full yield moment, $M_y = F_y(S_x)$, may be reached prior to local buckling.

"Slender Element sections" are those sections whose elements are subject to local buckling at a moment below the yield moment.

A reduction in the allowable bending stress is required for sections which are unstable, either laterally or torsionally, between their brace points. This is reflected in the Section F1.3, equations F1-6, F1-7, and F1-8. Since channels bent about their minor axis and loaded through their shear center are not subject to lateral-torsional buckling, equations F1-6, F1-7, and F1-8 are not applicable to them.

For "Compact sections" with shape factors, Z_y/S_y, greater than 1.10 AISC allows for a 10 percent increase in bending allowable, ($F_b = 0.66F_y$). Since the shape factor for most channels bent about their minor axis is in excess of 1.5, and the flanges of channels tend to be short and thick, nearly all "C" and "MC" channels will qualify as compact sections. Therefore, my recommendation is that channels bent about their minor axis should be designed with the following allowable stresses:

"Compact" channels bent about their minor axis and with shape factors in excess of 1.10, may be conservatively designed with an allowable bending stress of $F_{by} = 0.66F_y$.

"Non-compact" channels bent about their minor axis should be designed for $F_{by} = 0.6F_y$.

"Slender-Element" Channels bent about their minor axis should be designed for $F_{by} = 0.6(Q)F_y$.

Although justification exists for the use of $F_{by} = 0.75F_y$ for compact channels bent about their minor axis, as is done with wide flange sections, it is my recommendation that the more conservative compact section value of $F_{by} = 0.66F_y$ be used. Since channels are not doubly symmetric, the shape factor for channels bent about their minor axis tends to be more variable than for minor axis wide flange beams. The above is also consistent with allowable bending stresses for compact, non-compact, and slender elements given in the *Specification for Allowable Stress Design of Single-Angle Members*, Part 5 of the Manual.

William J. Bonefas, P.E.
H. G. Adams, Consulting Engineers
Fort Worth, TX

New Questions

Listed below are questions that we would like the readers to answer or discuss. If you have an answer or suggestion please send it to the Steel Interchange Editor, Modern Steel Construction, One East Wacker Dr., Suite 3100, Chicago, IL 60601-2001.

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

Are there special design rules and specifications for steel structures that will be in a "low" temperature area? Is the AISC *Specification for Structural Steel Buildings* appropriate for all temperatures?

What fatigue category should be used for a steel beam-to-column moment connection when the beam flanges have full-penetration welds to the column?

In a structure that has tubular columns, should weep holes be added at the bottom of the columns in order to drain any water in the column?

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Reviewing The Future Of Steel Design

By Nestor Iwankiw, AISC Director, Research & Codes

Almost all building and bridge designs have been, and continue to be, based on traditional elastic structural analysis. During the past decade or so, the additional direct evaluation of second-order effects, still within the elastic range, has gradually received more attention and use. However, the next progressive level of a complete second-order and inelastic analysis (also termed "advanced", "plastic", or "ultimate") continues to remain for most either a complete mystery or merely a side interest.

The major advantage of such an advanced analysis is that it can closely simulate the actual behavior and ultimate strength of a structure. With all the relevant strength, stability and serviceability limit states properly modeled, the computer analysis simultaneously becomes the design check. A separate verification of individual member or connection adequacy is thereby rendered unnecessary.

The origins of advanced analysis exist in the plastic design research conducted at Lehigh University in the 1950s and 1960s. Investigators have continued to research on this general topic related to questions on semi-rigid (partially restrained) construction, frame stability, and computer methods. Consequently, much has transpired since then. The status, goals and remaining needs of this work were brought into better focus by the Structural Stability Research Council (SSRC) with the formation of a new Task Group 29 on Second-Order Inelastic Analysis for Frame Design.

A new publication, *Plastic Hinge Based Methods for Advanced Analysis and Design of Steel Frames* (edited by Donald W. White and W.F. Chen) is a tangible product of this group's recent work and provides an assessment of the state-of-the-art. The book is a compendium of current technical papers totalling almost 300 pages by world-renowned researchers and consultants. All contributions have been subjected to a peer review by an expert panel and the careful scrutiny of editors White and Chen from Purdue University. The document is subdivided into three parts: Specification & Analysis; Practical Implementation & Use; and Verification & Benchmarking Problems. Each part contains ample material for education, additional research or design consideration.

The intrinsic nature of advanced analysis is theoretically and computationally more difficult than the usual linear elastic assumptions. Nevertheless, the promise of more accurate and realistic structural solutions is expected to render this the preferred method for the future, especially with the prevalence of computers and the growth of limit states design (LRFD).

For information on the $40 publication, contact SSRC at (610) 758-3522.

Correction

Flexural- Torsional Buckling of Structures is available through CRC Press, Inc. For ordering information call (800) 272-7737. We regret that this information was omitted from the review in the January 1994 issue.

Volume II—Connections

AISC has published errata for the *Manual of Steel Construction, Volume II—Connections, ASD 9th Ed./LRFD 1st Ed*. Most corrections are of an editorial nature, with the following exception: Tabulated values in the single-plate connection design aids printed on pages C-11 and C-15 (only) are incorrect; the correct values are given in the errata.

The errata will be mailed automatically to purchasers of this publication. Additionally, the errata will be printed in the 1st Quarter 1994 AISC Engineering Journal. If you purchased Volume II—Connections and did not receive the errata, call AISC at (312) 670-2400.

European Steel Market Statistics

Forecasts of steel usage in Europe, as well as historical data, are contained in a new publication from the European Convention for Constructional Steelwork (ECCS). The *1993 Statistical Bulletin* contains detailed information on construction activity in: Austria; Belgium; Croatia; Denmark; Finland; France; Germany; Italy, Netherlands; Norway; Spain, Sweden; Switzerland; and the United Kingdom.

A limited number of copies of this publication are available from the Technical and Research Department of AISC for $56 ($42 for AISC members). For information, call (312) 670-5411.

Steel Joist Vibration

A new computer program from the Steel Joist Institute is designed to help determine probable vibration characteristics of floor systems using open web steel joists.

Designed for use with SJI's Technical Digest #5 *Vibration of Steel Joist-Concrete Floor Slabs*, the program allows the designer to swiftly and easily calculate the frequency and amplitude resulting from transient vibration caused by human activity on a joist-concrete floor. The "what if" scenario—variations in slab thickness, concrete strength, joist size, joist spacing, floor decking, live and dead loads, span lengths—can be accomplished in seconds. Primary support systems consisting of joist girders or structural steel beams also can be analyzed as a part of the floor system.

The program can handle spans up to 100 ft. and costs $125 plus shipping and handling. For information, write: SJI, Suite A, 1205 48th Ave.. North, Myrtle Beach, SC 29577.

Better Contracts

A free 16-page primer from DPIC Companies is written to help architects and engineers draft effective contracts that minimize liabilities. *Introduction to Better Contracts* defines important terminology, analyzes contract formats, provides effective risk management tools, reveals tips on negotiating techniques, and lists "deal makers & deal breakers."

For a free copy, contact: DPIC Companies, P.O. Box DPIC, Monterey CA 93942 or fax 408/649-3240.

STEEL CALENDAR

An introduction to the new 1993 LRFD Specification and the *Manual of Steel Construction — LRFD, 2nd Edition* will highlight a new four-part seminar series from AISC Marketing, Inc. *Innovative Practices In Structural Steel* also will provide information on state-of-the-art structural steel design software, the latest NEHRP Seismic Regulations, and a review of semi-rigid composite connections.

The seven-hour, four-part seminar costs $90 ($75 for AISC members), including dinner. The lecture has a CEU value of 0.4.

For more information, call 312/670-2400 or fax 312/670-5403.

1994 Seminar Dates & Locations

WEST

Irvine	4/21
Sacramento	6/15
San Francisco	6/16
Los Angeles	6/23
Seattle	9/27
Salt Lake City	9/29
Phoenix	10/20
Portland, OR	11/15
Las Vegas	11/17

MIDWEST

Des Moines	5/4
Milwaukee	5/31
Minneapolis	6/2
Detroit	10/11
Indianapolis	10/13

NORTHEAST

New York	4/14
Albany	9/13
Rochester	9/14

SOUTHWEST

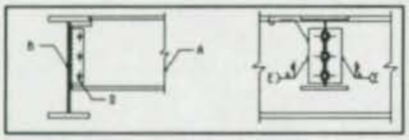
Dallas	4/19
Houston	6/21
Oklahoma City	9/8

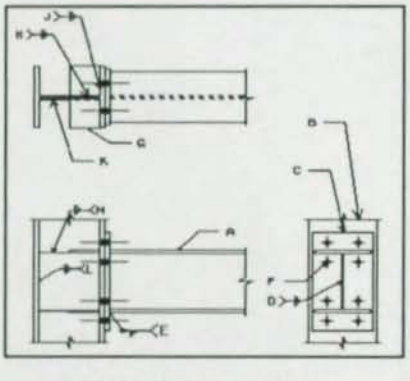
SOUTH

Birmingham	5/3
Atlanta	9/20
Richmond	9/22
Memphis	10/18
Miami	11/1
Orlando	11/3

MID-ATLANTIC

Baltimore	4/5
Washington, DC	4/7
Pittsburgh	4/12
Edison, NJ	10/4
Philadelphia	10/6
Cleveland	10/25
Columbus	10/26
Cincinnati	10/27





Other upcoming events related to the steel industry include:

•April 5 in Chicago—**SEAOI Meeting**. Featured will be a panel discussion on the safe erection of steel structures, including information on wind bracing, special connections and the coordination of temporary scaffolding. Contact: Barbara Pries at 312/372-4198.

•April 7 in Chicago—**SSPC Tutorials on Lead Paint Removal and complying with OSHA Lead Standards.** For information, contact: Megan McCormick at 412/687-1113.

•April 21 in Worthington, OH—**Steel Bridge Forum**. Steel Bridge Training Course on cost effective design and detailing. Call 202/452-7119.

•May 5 in Augusta, ME—**Steel Bridge Forum**. Steel Bridge Training Course on cost effective design and detailing. Call 202/452-7119.

•May 2, 4, 9, 11 & 16 in New York—**New Life For Old Structures: Rehab, Retrofit, Expansion** sponsored by New York Metropolitan Chapter of ASCE Structures Group. Contact: Eric Stovner at 212/741-1300.

•May 18-20 in Pittsburgh—**National Steel Construction Conference**. More than 20 seminars, technical sessions and workshops plus a 100-booth exhibition of steel-related products. Topics range from long-span structures to connection design. For more information, contact: AISC at 312/670-2400.

Earthquake Safety

The Northridge Earthquake once again demonstrated the inherent advantages of steel structures

This preliminary report was prepared by the Technology and Research Department of AISC.

After the Northridge Earthquake, newspapers, magazines, and television screens were filled with images of crumbled and collapsed structures—apartment buildings, bridges, parking structures, shopping centers and stores. What did almost every collapsed structure have in common? They were structures constructed of brittle materials such as concrete and masonry.

Little attention was given, however, to the excellent performance of steel structures in this seismic event. As Peter Yanev, chairman of EQE Engineering in San Francisco and a noted earthquake investigator, noted in a recent *Time* magazine article (Feb. 14, pg. 32): "It's quite simple: if you want to be safe in an earthquake, the best thing you can do is build in steel." Steel exhibits the properties that make it ideal for earthquake-resistant construction: a high-elastic limit, great plastic deformation capacity, and the internal strengthening mechanism of strain hardening.

Steel's Advantage

Steel's high elastic limit is important because steel structures are designed to meet building drift limits prescribed by building codes and to behave within the elastic range. Because elastic behavior means the steel structure will return to its original position after an earthquake, engineers can confidently predict its deflection during moderate earthquakes such as the Northridge event in Los Angeles.

Not all earthquakes, however, are moderate. Furthermore, seismic forces in an extreme earthquake will push the structure's behavior beyond the elastic range. When this happens, only a steel structure can deform plastically and dissipate the unanticipated energy imposed by the earthquake. From the viewpoint of life safety, the tremendous capability of steel to plastically deform is its most important asset.

Skilled engineers are striving to take full advantage of the inherent properties of steel. One of the critical design requirements is the use of proper connections. For areas of high seismicity, the AISC Committee on Specifications has developed a special standard called *Seismic Provisions for Structural Steel Buildings*. This document was prepared by a special task committee under the leadership of Professor Egor Popov of the University of California-Berkeley.

The task committee is continuing its work to further improve Seismic Provisions. Reports of local weld fractures in FR moment connections and damage to bracing members caused by the Northridge earthquake are being studied (see accompanying article). Based upon these new seismic experiences, AISC is ready to support a test program to establish modified design requirements if warranted. Nevertheless, it is a tribute to steel's amazing reserve strength and ductility that these steel structures continued to sustain loads and were in no danger of collapse in spite of these local problems. Furthermore, the necessary repairs, in most cases, have already been made.

In regions of lower seismic risk, however, special provisions are not needed; ordinary connection design procedures, as used for wind loading, are also applicable to seismic loads. In fact, in a keynote lecture at the 1991 AISC National Steel Construction Conference, Peter Yanev emphasized that steel-framed buildings not designed to seismic standards have often survived severe earthquakes with minimal damage.

Over and above the reserve strength and ductility inherent in steel as a material, current steel-frame design procedures are typically based on drift limitations. This results in an even greater overstrength in the steel frame itself. Research is underway at the University of California-San Diego by Professor Chia-Ming Uang to study how overstrength can simplify connection design.

Serviceability Considerations

Because achieving strong, earthquake-resistant structures is easy with steel, structural engineers pay special attention to serviceability requirements such as floor deflections and building drift. AISC has special design guides available to aid engineers in establishing design requirements for specific types of buildings; one example is the AISC *Design Guide Serviceability Design Considerations for Low-Rise Buildings*.

Concrete parking structures, such as the one pictured at top, were some of the hardest hit structures in the Northridge Earthquake (photo courtesy of the Earthquake Engineering Research Center). In contrast, most steel parking structures came through unscathed (photo courtesy of EQE Engineering).

For monumental high-rise structures, steel is the material of choice and serviceability can be assured through a variety of means. Some buildings, such as the John Hancock Building in Chicago, control lateral drift with bracing in the exterior frames. Other buildings, such as the World Trade Center Towers in New York City, use viscoelastic damping devices—engineer Leslie E. Robertson incorporated 20,000 simple viscoelastic dampers in the structural system of these towers that absorb the movements induced by unusual as well as average winds (*Engineering Journal*, Vol. 23, No. 4, 4th Quarter 1986). Others, including William LeMessurier, have employed passive tuned mass dampers to counteract movements. More recently, active mass dampers have been applied in Japan to mechanically counteract the effect of lateral forces.

Clearly, there are many ways to assure occupant comfort, but the fact remains that the basic steel structure is the key to life safety.

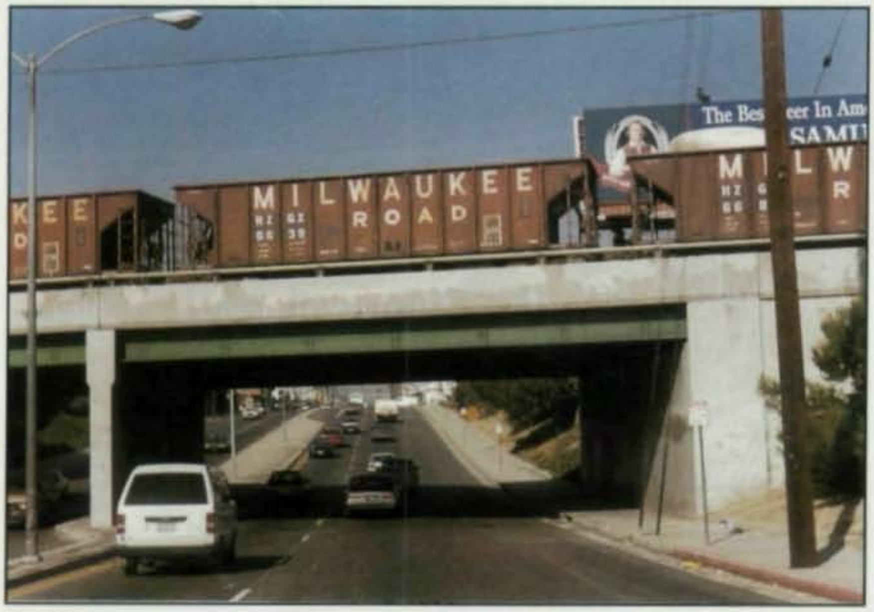
Case Study: Parking Structures

Some of the most devastated structures in the Northridge Earthquake were concrete-framed parking garages. Enough photos were shown in newspapers and magazines of the collapsed garage at Cal State to fill a book. That structure, along with many of the other precast concrete garages that were severely damaged, collapsed due to large horizontal and vertical ground motions that ruptured column-slab connections. As Peter Yanev explained in a recent talk on earthquake risk in St. Louis: "The problem with concrete garages is how to connect the columns and girders. You get something modeled on steel, but which never can perform like steel."

Yanev's point is best illustrated by four parking garages in a two-block area of Sherman Oaks. The three concrete garages, hard hit by the Northridge Earthquake, were closed indefinitely after several column failures and the partial collapse of both flooring and roof systems. The steel parking structure, which was also the oldest of the four, was undamaged and remained open.

So why aren't more parking structures in Southern California built in steel? One problem is that some people incorrectly perceive that steel parking structures have a floor vibration problem. This school of thought contends that even though steel parking structures are structurally sound—even in an earthquake—people don't perceive them to be that way.

While floor vibrations may have been a problem 20 years



Damage to concrete overpasses of roadways such as Interstate 5 have played havoc with ground traffic in Southern California (top photo courtesy of Earthquake Engineering Research Center). However, steel railroad bridges, such as this one in Northridge, performed exceptionally well (photo courtesy of AISC Marketing).

ago, a number of developments have remedied the situation. Research by Professor Thomas M. Murray of Virginia Tech, among others, has resulted in new procedures and design methods to minimize vibration problems.

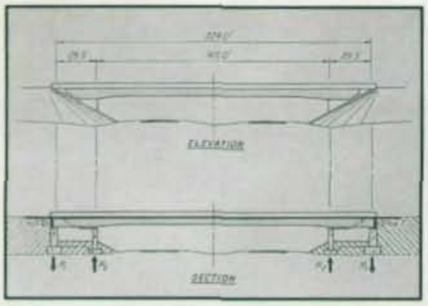
In other parts of the country, older parking structures (both steel-framed and concrete-framed) have experienced deterioration of the concrete deck due to de-icing salts. However, AISC's Design Guide *Designing Open Deck Parking Structures* provides information on how to prevent this damage. Modern designs have been so successful that in the Northeast, especially Massachusetts, steel is the dominant material for multi-level above-ground parking decks.

Case Study: Bridges

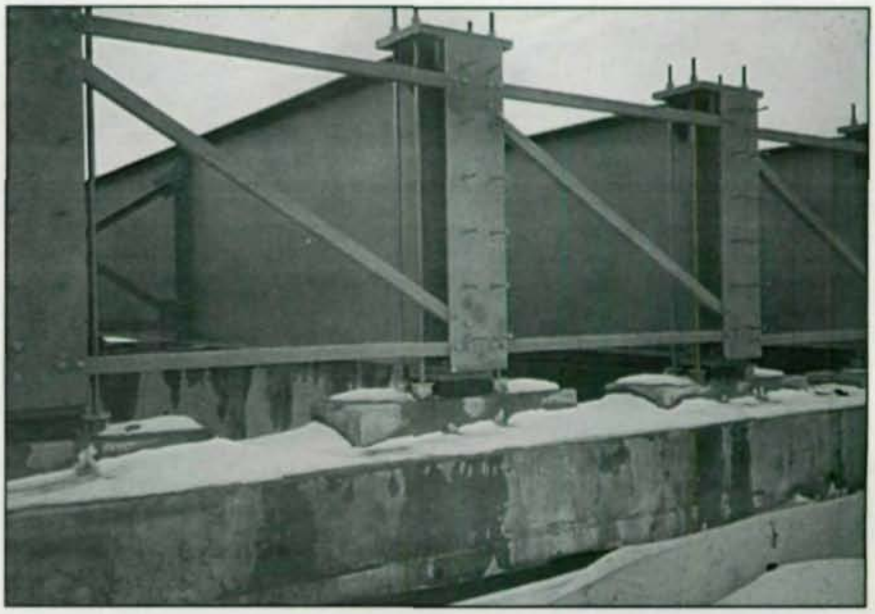
The biggest problem for most people after the Northridge Earthquake was the damage to roads and bridges. This was also the case with the San Fernando Valley Earthquake of 1971, which affected much of the same area. In fact, some of the bridges that collapsed this year were along the same roadway and in the same area as bridges that collapsed in 1971.

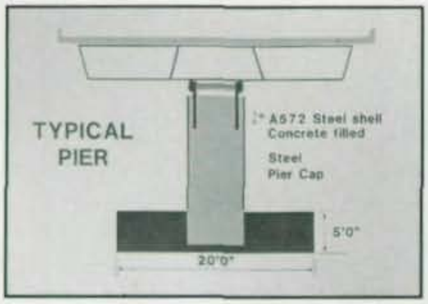
An example of the type of damage that occurred along I-5, Interstate 10, Route 118, Route 405 and Route 101 was what happened at the Interstate 5/Route 14 Interchange. The interchange, which consists primarily of concrete box girders supported by single column bents, was devastated by the earthquake. Damage included the collapse of the eastern end frame of the North Connector and the collapse of the southern end frame of the South Overhead. Referring to the North Connector failure, the *Preliminary Report on the Seismological and Engineering Aspects of the January 17, 1994 Northridge Earthquake* from the Earthquake Engineering Research Center (EERC) at the University of California at Berkeley stated: "The simple span fell off the seat abutment, but the transverse shear keys remained intact. A shear failure in the bent 2 column appears to have initiated the collapse."

Ironically, steel bridges are being built in non-seismic areas that would easily withstand seismic forces without major damage. A good example is the series of anchored end-span bridges recently completed by the Illinois Department of Transportation for a new interstate highway (I-39) from Springfield to Wisconsin. The ends of the girders are anchored to the abutments with prestressed rods, eliminating the need for any expansion joints. In Illinois, this detail is important because it eliminates any chance for



Anchored end-span bridges, such as those along Interstate 39 in Illinois, are designed to eliminate salt corrosion problems. However, this type of design would also be useful in seismic areas.





The trapezoidal composite box girders of the Melrose Interchange near Memphis are connected to the tops of the piers with integral pier caps. Pin connections prevent the bridge from sliding off the pier—a useful feature during an earthquake.

salt-laden water to reach the girders, which is a common cause of corrosion problems in older bridges. In a seismic area, that same detail would prove invaluable in preventing the type of collapse that occurred on some of the bridges in California. An added benefit is the elimination of mid-span pier supports—a safety bonus for the driving public. End-anchored bridges have also been constructed in Colorado and Tennessee.

The Melrose Interchange near Nashville, Tennessee, is another superb example of a well-designed steel bridge. Trapezoidal composite box girders are connected to the tops of the piers with integral pier caps; pin connections prevent the bridge from sliding off the pier. The piers consist of welded steel circular shells filled with concrete. Again, this type of construction would prevent the concrete from spalling in the event of an earthquake.

Tennessee also is a leader in applying a new method of steel bridge design called "autostress," which is based on the ability of steel to yield and automatically redistribute its loads. A demonstration of steel's ability to withstand unusual and unpredictable loadings occurred during the construction of the Obion River Bridge. As described in a paper given at the 1992 Transportation Research Board Meeting by Edward Wasserman, P.E., Civil Engineer, Director of Structures with the Tennessee State Department of Transportation, upon nearing completion, one of the piers of this nine-span bridge began moving due to a foundation problem. However, the bridge did not collapse even though one pier was completely removed. Since there was no disastrous collapse, photographs of this dramatic behavior never received attention in the popular media. Other states, including

All-steel bridges are extremely common in earthquake-prone Japan. The Higashi-Kobe Bridge (above and above left) is an all-steel, cable-stayed bridge with steel towers, steel trusses and two levels of steel plate decks.

Many Japanese bridges, such as the one pictured at left, have two levels. The use of steel frames helps to resist seismic forces and prevents the pancaking that occurred with the two-level Cypress Viaduct in Oakland during the 1989 Loma Prieta Earthquake.

New York and Maine, also are using this method.

Designers in other countries also have recognized the value of steel for bridge design. Whereas most steel bridges are in reality composite steel bridges, many all-steel bridges have been and continue to be built in Japan, where earthquakes are even more common than in California. The Higashi-Kobe Bridge is an all-steel, cable-stayed bridge with steel towers, steel trusses, and two levels of steel plate decks. The only non-steel element is the asphalt wearing course over the bridge decks.

Two-level expressways also are common in Japan, where they are designed as steel frames to resist strong earthquakes and prevent the pancaking that occurred with the Cypress Viaduct in Oakland during the Loma Prieta Earthquake in 1989.

Case Study: Hospitals

The Northridge Earthquake played havoc with many of the area's health care facilities. Several concrete hospitals were closed for extended periods of time and suffered substantial damage. For example, repairing the severely damaged St. Johns Hospital in Santa Monica is expected to cost approximately $50 million. Likewise, the Indian Hills Hospital suffered very serious structural and non-structural damage. According to the EERC Preliminary Report: "The Indian Hills Hospital suffered structural damage in the shear walls with concrete crushing and apparent lap splice failure at the construction joint at the fourth floor level."

Again, this unfortunately paralleled the events of the 1971 earthquake. However, at least one facility did learn its lesson.

In 1971, the Olive View Medical Center at Sylmar, California, a concrete-framed structure, was totally destroyed. At that time, the hospital administrators vowed to never again let an earthquake devastate their facility. As a result, when they rebuilt, they opted for a building system featuring a steel frame and steel plate shear walls.

The new Olive View Medical Center is approximately eight miles from the epicenter of the Northridge Earthquake and roof top accelerometers measured a tremendous horizontal acceleration of 2.3 g. However, while the sprinkler system and a roof-top air conditioner sustained minor damage, the facility remained completely operational. There weren't even any broken windows. As Peter Yanev stated when he viewed the facility: "That hospital didn't make the same mistake twice."

Another hospital that fared well was the University of Southern California Teaching Hospital. The hospital is an 8-story braced steel frame supported on 68 lead-rubber isolators and 81 elastomeric isolators. It is located east of downtown L.A., about 24 miles from the earthquake's epicenter. Despite its distance, peak free-field acceleration reached 0.49 g, peak foundation acceleration was 0.37 g, while peak structure accelerations were 0.13 g and 0.21 g at the base and roof, respectively. According to the EERC Preliminary Report: "The hospital remained completely functional during and after the earthquake, and there were no reports of damage to equipment inside the building." Other steel-framed buildings utilizing base isolation systems reported similar successes.

After the Olive View Medical Center collapsed during the Sylmar Earthquake in 1971 (top), it was rebuilt in steel (shown above after the Northridge Earthquake). Photos courtesy of EQE Engineering.

Conclusions

Because steel-framed construction minimizes the risk of a catastrophic failure, steel is the obvious material of choice for the design of earthquake-resistant buildings, bridges, and parking structures. It offers strength and ductility to resist major earthquakes without damage and provides the utmost in reserve strength, ductility, and overstrength in severe seismic events. Additionally, earthquake damage in steel structures is usually localized; any necessary repairs can be made relatively quickly to restore the structure to normal service. Ultimately, no other material offers a better opportunity for the preservation of life safety.

All-steel bridge construction, as practiced in Japan, represents the best possible seismic resistance and maximum longevity with proper maintenance.

AISC's *Seismic Provisions for Structural Steel Buildings* contributes to the knowledge base needed by engineers for the design of steel frames and connections with the required ductility. Additionally, good steel-frame design practice in regions of lower seismic risk provides automatic seismic resistance even when this is not a code requirement.

Localized Steel Damage

A small number of steel-framed buildings experienced localized weld failure during the Northridge Earthquake

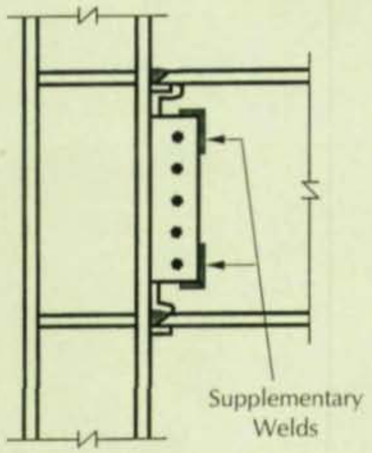
At least seven steel-framed structures, and perhaps as many as a dozen, had localized connection failures during the Northridge Earthquake. The localized problems—which did not cause a collapse or a life-safety concern in the structures—occurred primarily in recently constructed low- and mid-rise structures with fully restrained moment connections.

There were two broad problems, according to Michael Engelhardt, an assistant professor of civil engineering at the University of Texas. The more common problem was a weld fracture, which can probably be corrected by improving the weld or changing the connection detail to reduce stress on the weld.

A more serious problem, however, is the column flange cracking that occurred in some of the affected buildings, Engelhardt said. In some cases, these fractures occurred through the thickness of the flange and their cause is as yet unknown. "It was a fracture type problem and will require some study to resolve," Engelhardt said. Many of the fractures occurred between the beam flange and the continuity plate.

More easily resolved are the weld failures. Typical of the problem was a four-story, four-year-old office building that came through the earthquake with no apparent damage. However, afterwards, during routine tenant improvement work, the damage to the welds was discovered. The roughly 100,000-sq.-ft. structure is a fully restrained moment frame with bolted web connections and welded flanges. Because the

Welded Flange - Bolted Web Detail

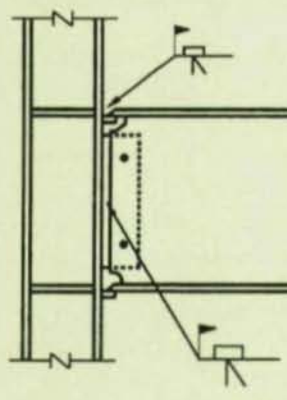


Current U.S. Code Requirement

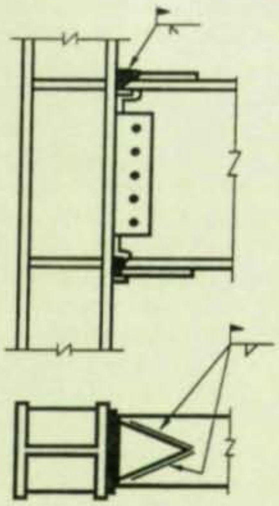
If Z_f / Z is less than 0.7:
Provide supplementary welds with strength to develop 20% of $M_{p\,web}$

If Z_f / Z is greater than or equal to 0.7:
No supplementary welds required

Proposed Solution 1:
All-Welded Moment Connection



Proposed Solution 2:
Reinforced Moment Connection



flanges alone provided more than 70% of the plastic moment of the beam, welds were not required in the web connection according to the *Seismic Provisions*.

"We found an occurrence rate of approximately 10-15%," stated Thomas Sabol, S.E., president of Englekirk & Sabol, Inc., a Los Angeles engineering firm investigating some of the localized failures. "It appeared to be a fracture of the welds; we didn't see any plastic deformation of the girder." The problem is being repaired by grinding out the damaged weld material and re-welding. In addition, some continuity plates are being added and beams are being reinforced.

"In essence, what we're doing is thickening up the flanges of the beam by welding on a triangular-shaped plate." It's the same width as the beam flange and then tapers to a point to avoid any stress concentrations. The thickness is sized to reduce the stress in the full penetration weld to the column; the length is sized to avoid block shear and to provide enough metal so the plate does not get pulled off.

Another possible solution would be to weld the web connections.

"The problem is not life-threatening, but it is serious because steel ductile frames are the premier system for seismic areas and were thought to be very reliable systems," Sabol said. While this is usually true, in these few cases a problem developed. "The steel industry will have to give a lot of thought on how to revise their details to correct this problem," he added. A steel industry task committee, made up of industry representatives, practicing engineers and researchers, met in mid-March but their results were not available at presstime.

The weld cracking occurred in either the top and bottom flanges or both of approximately 20% of the connections in one moment-framed building examined by Nabih Youseff, S.E., principal of Nabih Youseff & Associates in Los Angeles. The problem, he reported, may be that this earthquake had more of a "shock effect" while most building connections are designed for cyclic loading. In contrast, nuclear facilities and bridges are both designed for shock effects (e.g., a freeze-thaw event).

He also noted that the problem may be related to recent advances made in connection design. "The components of the the joint may have more ductility than the weld material," he theorized. The problem is probably not related to workmanship because of its consistent occurrence, he added.

Flange Cracking

The problem also occurred on a six-story building that was just nearing completion. Again, this moment-framed structure had bolted webs and welded flanges. As with the other building, the web connections did not need to be welded. The problem was discovered when hairline cracks showed up in the fireproofing on the connections. When the fireproofing was stripped, the damaged connections were revealed. However, unlike some of the other affected buildings, in addition to the localized weld failures this structure also had some cracked column flanges. "There were horizontal cracks right above the top of the bottom flange of the beam," Sabol said. Some of the flanges were removed for testing, but results are not yet available.

Cracking also was a problem on a two-year-old, four-story office building. "The structure is an all-welded ductile frame on four sides," according to Elwood Smietana, S.E., a vice president with EQE Engineering's Los Angeles regional office and one of the engineers investigating the problem. "Most connections at the first level and many at the upper levels had a problem." While no collapse was imminent, reserve capacity in the structure was estimated at significantly less than half of the pre-earthquake level. "There was flange and web cracking outside and inside the connection," he added. One weld expert who examined the structure said that he believes that insufficient preheat for some of the beam-column welds may have contributed to the failure, Smietana reported. Some experts also theorized that the weld failure may have initiated the flange cracking.

A weld procedure is currently being qualified for the repair of the structure, which could run approximately 20% of the building's value.

According to one expert, the steel industry may have to review changes in details made during the past decade. For example, the move towards less redundancy and thicker columns could be a problem, as could the increased use of high-speed, self-shielded flux core, though he cautioned that more study is needed.

A localized weld failure also occurred on a chevron-braced two-story office building over one-story of underground parking. "The weld between the tube brace and the gusset plate had fractured and the braces deflected out of plane," Sabol explained. The building remained occupied and the problem was quickly repaired in a few weeks. The contractor ground out the weld and fillet-welded all the way around the plate. In addition, the centers of some of the braces were reinforced.

Part of the reason for damage, according to Egor Popov, professor emeritus at the University of California at Berkeley and chairman of the AISC Specification Task Committee on Seismic Design, is that building codes in the U.S. are designed for life safety, not necessarily to prevent any damage from occurring to a building. In that regard, Popov pointed out, the buildings performed well, with none in any danger of collapsing.

Lessons Learned From The Northridge Earthquake

The midwest and northeast portions of the U.S. are vulnerable to earthquakes and should take heed of California's experience

Damage to concrete overpasses in 1971 (top) was similar to the damage suffered in 1994 (above). Photos courtesy of EQE Engineering.

The Northridge Earthquake was the second most costly natural disaster in U.S. history, trailing behind only Hurricane Andrew, according to one of the country's leading structural investigators. What made the earthquake more devastating, however, was that only about 10% of the estimated $20 billion damage in California was covered by insurance, while 60% of the $25 billion in damage caused by the hurricane was covered.

Because the epicenter of the Northridge Earthquake was in a heavily built-up area, there was three to four times as much damage as in San Francisco during the Loma Prieta Earthquake in 1989, according to Peter Yanev, S.E., chairman of EQE International, San Francisco. Yanev spoke recently in St. Louis on this year's earthquake and the need for the midwest to prepare for a similar disaster with the New Madrid fault. "You'll get an earthquake sooner or later," Yanev commented. "And you'll have the same problems as in California."

According to initial estimates, nearly one-quarter of the damage occurred to buildings, the same amount to lifelines (power systems, utilities, roads, bridges, telephone systems, etc.), almost a third to residences, 16% to industrial facilities, and 4% to government buildings.

Interestingly, much of the damage paralleled the destruction caused by the 1971 San Fernando Earthquake—and some of that was due to owners and developers repeating some of the same mistakes. For example, when the concrete I-5 overpasses were severely damaged in 1971, they were rebuilt in concrete, Yanev pointed out. This year, additional concrete overpasses on I-5 collapsed.

In comparison, the management of Olive View Hospital took a smarter approach. When their concrete hospital was damaged beyond repair by the 1971 earthquake, a new steel-framed hospital was constructed. With the exception of some damage to rooftop mechanical equipment and sprinkler pipes, the new Olive View Hospital emerged unscathed. "That hospital didn't make the same mistake twice," Yanev noted. He expressed optimism that other hospitals that were hard hit in the 1994 earthquake, such as the concrete St. Johns Hospital in Santa Monica that was hit with an estimated $50 million in damage, would learn from Olive View and rebuild in steel.

Much of Yanev's talk focused on what was damaged, and what wasn't. As with earlier earthquakes both here and abroad, the popular press did a good job covering death and destruction but ignored the thousands of buildings—many of them steel framed—that survived with no damage, he noted.

By far the hardest hit construction technology was unreinforced masonry and tilt-up concrete buildings. "Almost one-third of all the unreinforced masonry and tilt-up buildings in the valley had severe damage—the same problem that we saw in 1971," he stated.

Localized Steel Damage

In contrast, most steel-framed buildings came through the earthquake relatively undamaged. Some steel buildings had non-structural motion damage, but on most buildings where the interior had been brought up to code there was no damage, he noted. Likewise, better preparation would have spared the rooftop mechanical equipment that was damaged on some steel-framed buildings. Structural damage to steel-framed buildings was limited primarily to some localized weld failures that were quickly and easily repaired (see accompanying story).

While a concrete garage in the Sherman Oaks area was substantially damaged by the Northridge Earthquake and needed to be closed, an older steel garage immediately across the street remained open. Photo courtesy of EQE Engineering.

Some of the most spectacular damage occurred to parking structures and shopping centers, according to Yanev. Two structures in particular—the Northridge Mall and the parking garage at Cal-State Northridge—received massive media attention.

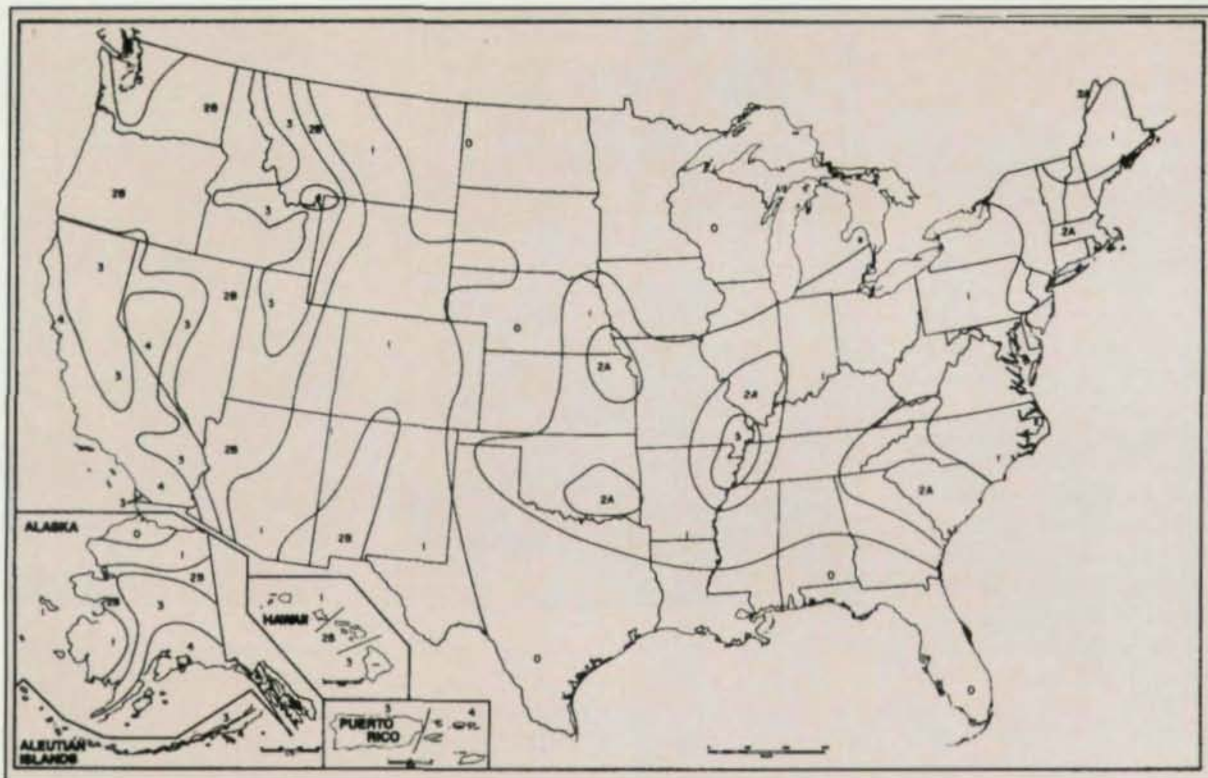
The Northridge Mall was notable not only for the damage it sustained, but to the economic impact its closing will have on the neighboring community. Retail stores in this area generated an estimated $377 million per year in revenues. The biggest failure was the Bullocks Department Store, which featured concrete-frame construction without shear walls. "This type of construction was typical of the 1950s through 1970s in California, and is very common in much of the Midwest," according to Yanev. During the earthquake, the entire interior of the structure collapsed, primarily because the three-story-high columns were not capable of supporting gravity loads under seismic forces.

Shear wall concrete construction at the mall fared much better, though there was still extensive structural damage. The Broadway store, which featured continuous shear wall concrete construction did not collapse and may re-open before Christmas.

In contrast, the only steel-framed building at the mall, a small retail building, came through the earthquake without structural damage. However, it did sustain a lot of interior finish damage. "A lot of the light fixtures came down—something that could have been prevented at a cost of about 10 cents per sq. ft."

Newer Failures

The parking structure at Cal-State Northridge received a lot of media attention because of



U.S. Seismic Zones according to UBC

the extent of its damage. But for engineers, the failure of that parking structure was notable because of its age—it had been completed only 18 months before the earthquake.

The parking garage was designed according to the Uniform Building Code for seismic zone 4 construction. The exterior columns were designed to carry the entire load; therefore the interior columns did not have to resist seismic loads, according to Yanev. Because of the vertical forces of this earthquake, the gravity system of this structure failed and the structure collapsed. "A steel-framed building would not have fallen down," Yanev asserted.

Other concrete parking structures had similar problems, he added. Yanev stated that he was particularly taken by the scene he surveyed in the Sherman Oaks area. "In an area with many garages, only the steel garage was still functioning immediately after the earthquake." (For more information on parking structures, see accompanying story.)

Yanev has long been a strong proponent of steel construction in seismic areas. In 1974, he authored "Peace of Mind in Earthquake Country," a well-received book detailing the problems facing California's built environment. Based on his research for that book, he began exposing the hazards of unreinforced concrete, masonry and tilt-up construction in seismic areas. "If you want to design in concrete, at least use shear walls," he recently stated. The book even discussed the possibility of an earthquake in the Northridge area.

Warning For The Midwest

Lately, he has been warning parts of the U.S. outside of California against complacency, a point hammered home by a minor earthquake (approximately M4.4) outside of St. Louis just a few days before his talk. Based on historical data, the northeast, southeast and midwest are all vulnerable to earthquakes.

For existing buildings, Yanev suggests hiring a qualified engineer to assess the risk and, if necessary, prepare a retrofit plan. Retrofitting older buildings can be particularly cost saving since even minor damage could expose asbestos, which could lead to a very costly abatement plan. "Asbestos has to be an issue with building cleanup and earthquake design," Yanev stressed. In the Northridge Earthquake, most retrofitted buildings performed quite well and experienced minimal damage.

For new construction, it is important that the building is designed to resist seismic forces. "Generally, the more steel the better," he said.

In addition, it is important to ensure that interior elements—mechanical equipment, inventory storage, file cabinets, lighting fixtures, fire protection equipment, computers, etc.—are adequately braced to come through an earthquake undamaged. This short-term cost is relatively low compared with the long-term cost of closing an operation for an indefinite period of time.

Yanev ended his presentation with a picture of a large circus tent. "That's the new headquarters for an unprepared company in Northridge."

Parking Problems

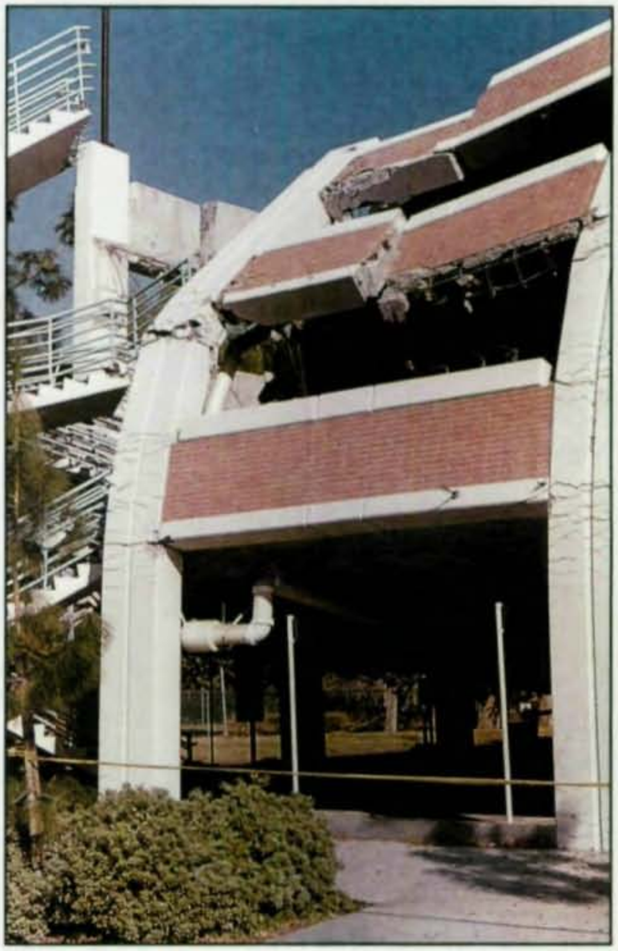
Parking garages fared the worst of all non-residential structures during the Northridge Earthquake

"I've been telling my wife for years not to park in [precast] garages like that. It's almost a joke in the family." — Peter Yanev, chairman, EQE Engineering, as quoted in the Los Angeles Times.

At least eight public parking structures—half less than six years old and one less than two—partially or completely collapsed during the Northridge Earthquake. "Parking structures represent the category of modern engineered structures that appear to have suffered the largest incidence of partial or total collapse cases," according to a preliminary report issued by the Earthquake Engineering Research Center at the University of California at Berkeley.

"Most cases of partial or complete collapse involve modern precast parking structures which either lack a lateral load resisting system in one direction or, otherwise, have a very flexible lateral load resisting system in one or both directions," the report continued. "Several such structures virtually 'imploded'...."

The most dramatic failure was undoubtedly the parking structure at the California State University at Northridge, an $11.5 million, 2,500-car garage built less than two years ago. According to W. Gene Corley, S.E., a vice president with Construction Technology Laboratories, Inc., Skokie, IL, and chairman of the ACI Building Code Committee, the garage experienced a "partial collapse caused by the gravity load system in the building." Corley, who examined a number of garage structures immediately after the earthquake, reported that the Cal State garage featured a perimeter lateral load resisting system with the interior columns only designed to resist gravity loads. "A combination of a high 1.2 g vertical acceleration combined with loads thrown into the gravity load system resulted in the interior columns being overloaded and precipitated a collapse," he theorized. The center was less damaged, and Corley said he believes it could conceivably be repaired, with the exterior bays being removed and replaced.



Although completed only 18 months before the Northridge Earthquake, a 2,500-car parking structure at California State University virtually imploded. Photo courtesy of AISC Marketing.

The garage was constructed of precast concrete (mostly site precast) and was constructed in accordance with the latest Uniform Building Code requirements. The actual damage consisted of the complete collapse of three bays at one end, as well as several bays at the other end.

Understanding The Collapse

"Possible causes of such total collapse [of the Cal State and other garages] might be the unseating of the precast girders due to large lateral movement at the short corbel seats or the shear-compression failure of the columns," stated the EERC preliminary report. "In all cases the prestressing tendons in the floor slab provided a catenary action that caused the spectacular "implosion" of part or all of the

continued on page 30

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TO ANNOUNCE A R

Arched chord joists top the Arrivals Hall, International Concourse, Hartsfield Atlanta International Airport

Parking garages in the northeast and midwest have successfully solved problems with serviceability and salt corrosion. Pictured above is a 628-car municipal garage in East Lansing, MI, while shown at left is a 550-car parking structure in Patterson, N.J.,

structure. Other areas of weakness appear to be the connections of precast girders to the corbel seats at the columns. These connections commonly involve the welding of a plate at the bottom of the girder to an angle at the free corner of the corbel. Weld failures were observed in the post-earthquake survey of damage, as was the 'chipping-off' of the corner of the corbel that reduced the seating area of the precast girder. The latter cause could have precipitated the unseating of the precast girder particularly under the high vertical accelerations."

Or, as Peter Yanev eloquently put it: "The problem is you get something modeled on steel but which can never perform like steel."

Other Problems

"Another area of weakness in modern precast parking structures is the flexibility of the thin cast-in-place topping slab that forms the horizontal floor and roof diaphragms," according to the EERC preliminary report. "Significant compression crushing was evident in the roof diaphragm of the City Hall Parking Structure, where the addition of another parking floor with insufficient lateral load resistance appears to be the cause of the partial roof collapse. The falling debris from the supporting beam and a planter punched through two floors of the three-story parking structure." One final problem noted in the report concerned the shear cracking in the columns of some parking structures.

Other concrete garages suffering substantial damage include: Northridge Fashion Center, north parking structure (750 cars); Northridge Fashion Center, south parking structure (650 cars); Sherman Oaks Fashion Square, south garage; Trans World Bank (150 cars); Glendale Fashion Center (878 cars); Glendale Civic Center (563 cars); and Kaiser Permanente West Los Angeles Medical Center (390 cars).

Both Yanev and Corley noted that the Sherman Oaks Fashion Square and Trans World Bank garages are in close proximity to an older (pre-1980) steel garage that came through without any damage. "There are four garages located close to each other, and only the steel structure was still operating immediately after the earthquake—and it was the oldest of the four," Yanev stated.

Why Not Steel?

According to Corley, designers in California have moved away from steel parking structures primarily because of perceived floor vibration problems. However, parking designers on the east coast (see September-October 1990 and January 1993 *Modern Steel Construction*) have long since learned how to minimize vibrations and steel parking structures are built extensively in New England without any user complaints about "bounciness".

Another falsely perceived problem with steel parking structures is the requirement for fireproofing. In fact, most current building codes allow multiple-story, steel parking structures to be constructed without fireproofing as long as at least two sides are 50% open and exit conditions are met.

A final concern is not related to parking structure design in California, but rather to designs in the snow-belt. In many areas, de-icing salt has led to deck deterioration. This is actually a problem common to both steel and concrete garages, and AISC has issued a Design Guide (*Designing Open Deck Parking Structures*) that discusses how to prevent this damage.

Traditional Values

The new home of the Texas Rangers uses structural steel to capture an "old-time" look

By David A. Platten, P.E.

Inside and out, the new home for the Texas Rangers reflects traditional stadium design.

Located midway between Dallas and Fort Worth, The Ballpark in Arlington is ready to open the 1994 baseball season. The major league ballpark, which seats 48,100, is the focal point of a $165 million project that includes a little league ballpark, an outdoor amphitheater, festival retail, and a series of man-made lakes.

The ballpark structure measures 850-ft.-by-850-ft. in plan. Exterior elevations consist of a series of large arches detailed with red brick and precast above a series of smaller arches clad in pink granite. The facade is punctuated with cast stone Texas icons, including Texas stars and steer heads. The majority of the seats are distributed among three decks, the lower deck, loge, and upper deck. Additionally, a covered "home run porch" beyond right field holds 6,000 spectators. Two levels of luxury suites are provided, one between the lower deck and the loge and the second between the loge and upper deck.

Concessions and restrooms are located on three concourse levels. The main concourse is located at grade and serves the lower seating deck. The club concourse is 39-ft. above the main concourse and serves the loge level as well as the suite levels above and below. The upper concourse is 70-ft. above the main concourse and services the upper seating deck. All concourse and suite levels are accessed by ramps, escalators and elevators. Team clubhouses, grounds keeping facilities, and other support functions are located at playing level, 20-ft. below the main concourse.

The playing field consists of natural grass with an asymmetrical outfield configuration. Outfield dimensions range from 325-ft. in the right field corner to 403-ft. in right-center field. Bullpens are located beyond the outfield, and are elevated for better sightlines. A four-level office building located beyond center field houses club offices.

Owner's Goals

From the outset, the Rangers ownership had a clear vision of what they wanted their new home to be. They wanted an open-air environment, natural grass and an "old-time" feel—but with modern amenities. From the outside, it needed to be distinctly Texan. Inside, the seating needed to be configured to minimize the distance between spectators and the playing field. And, given the blazing Texas summer sun, seating and public areas needed to be shaded as much as possible.

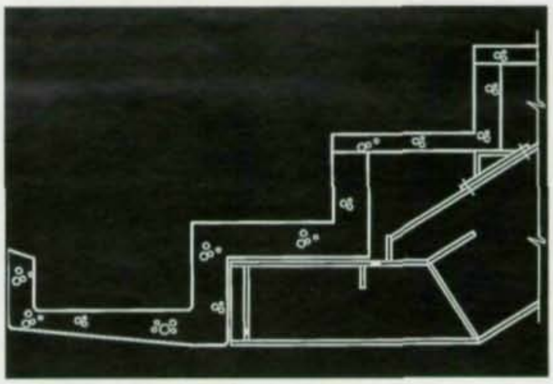
To achieve the desired "old-time" look, the Rangers expressed an interest in a steel structural framing system. As a result, a typical ballpark cross section was developed utilizing structural steel with precast concrete seating units. However, to provide an economic comparison, a cast-in-place concrete cross section also was developed. (An all precast concrete structural frame was undesirable to the ownership, and therefore was not considered.)

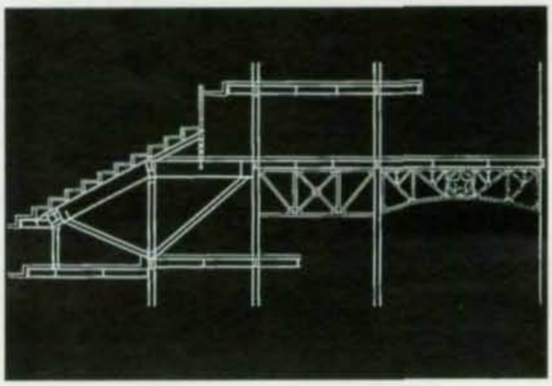
The construction manager priced the structural steel superstructure vs. the cast-in-place concrete frame and steel was chosen for the following reasons:

1. The desired "old-time" look of "steel and rivets" could be achieved.

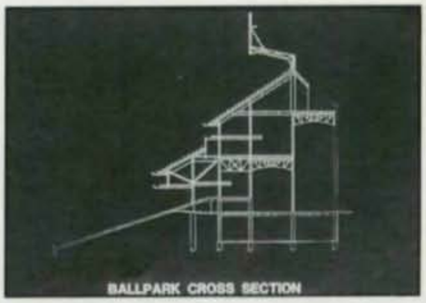
2. The cost of the two systems was comparable in the areas of the ballpark that had floor-to-floor heights of 15-ft. or less. However, cast-in-place concrete was cost-prohibitive for the upper concourse, located 70-ft. above the main concourse.

3. Construction time was criti-

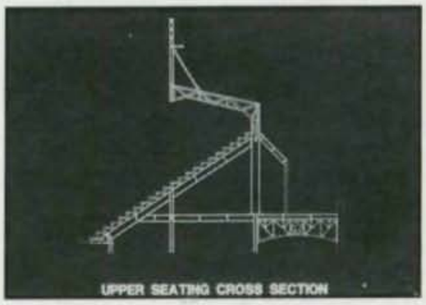




Shown above is the structural frame, looking down the firstbase side from the loge seating. The series of details at left show, from top to bottom: the connection of the precast seating to the steel; a close-up photograph of the connection; and a loge seating cross section.



BALLPARK CROSS SECTION



UPPER SEATING CROSS SECTION

cal and a steel stadium could be erected much faster than a cast-in-place structure.

Framing System

Foundations consisted of more than 700 drilled piers 24-to 48-in. in diameter founded in shale 20-to 30-ft. below the service level elevation. Existing grade sloped 20-to 25-ft. across the site, allowing for a balance of cut and fill on site with excavation requirements minimized to the greatest extent possible. To allow construction to begin while structural steel and precast seating units were being detailed and fabricated, the main concourse level was framed in reinforced concrete, with concrete columns, shear walls, and basement walls extending from the service level 20-ft. below.

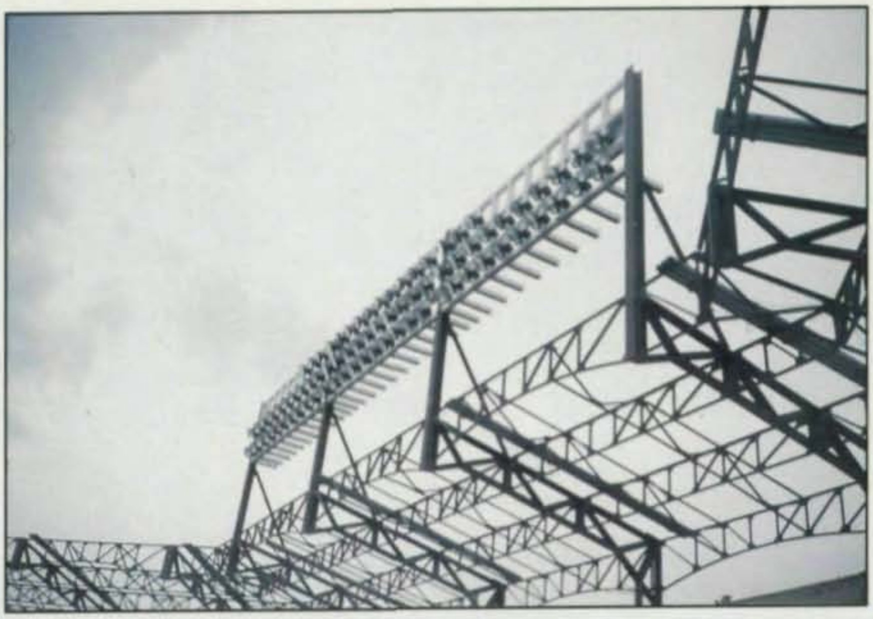
The lower seating bowl was constructed on engineered fill, which was available on-site. Structural steel column bases occurred at the lower seating/main concourse level. Levels above were framed with 7,500 tons of A36 and A572 Grade 50 structural steel. Primary structural steel frames and seating bents were located 32-ft.-8-in. on center. Double-tread precast seating units span the 32-ft.-8-in. dimension to the structural steel frames. Story-deep trusses allow the lower suite level and loge seating to cantilever over the rear portion of the lower seating bowl. A 30-ft. canopy cantilevers from the columns located behind the upper seating level, providing shade over a large portion of the upper deck. Trusses spanning 60-ft. provide a roof over the "home run porch" beyond right field.

Framing that occurred within enclosed luxury suites and concourse areas received sprayed-on fireproofing. However, columns that supported these areas were typically exposed at the main concourse level below. Spraying these columns was an issue. To address this problem, a series of life-safety analyses were performed that predicted column temperatures during a fire based on the nature and extent of combustibles located in various areas of the main concourse. Analytical results demonstrated that if minimum web and flange thicknesses equivalent to a W14x90 section were provided, temperatures reached during the worst-case fire conditions would remain below critical levels.

Lateral Load Resistance

Due to the large plan dimensions of the exposed structure, expansion joints were introduced to control the build-up of thermal stresses. Double lines of beams and columns were utilized at the one-third points along each side of the structure, resulting in a maximum expansion joint spacing of approximately 280-ft. Lateral loads are resisted within each section of the structure by X-braced frames, which contain 1-1/8-in. A490 slip critical bolted connections.

Wind tunnel tests were conducted on a variety of factors and proved very interesting. Wind studies of the flight of the ball proved interesting, and ultimately had a significant architectural impact. Early project designs consisted of only a two level office building beyond center field, as well as a much more



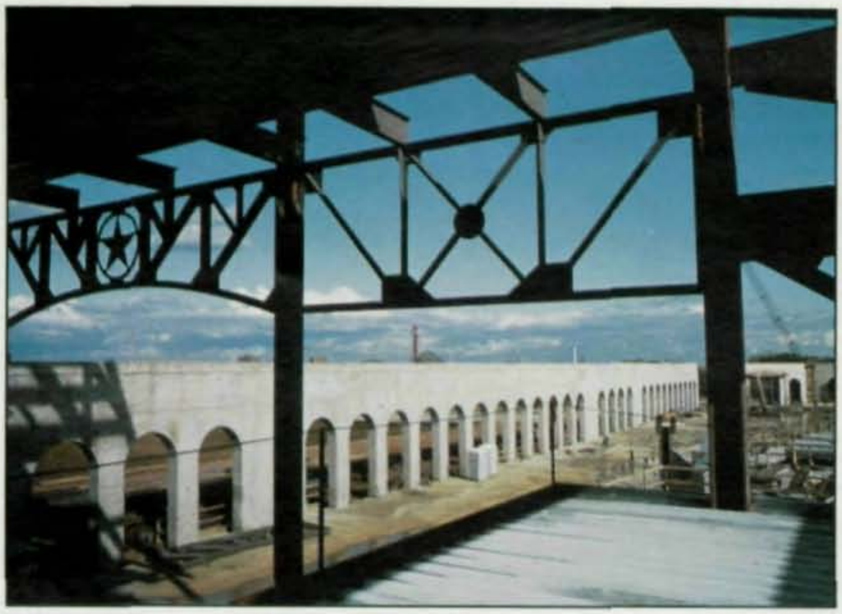
The canopy structure above the upper deck has main cantilevered trusses at 32'-8" on center, with secondary trusses spanning between. Vierendeel lighting trusses are supported from the ends of the main cantilevered trusses. Pictured opposite is a view of the flat-bottom truss adjacent to the curved-bottom truss supporting the Club Concourse framing.

open configuration in the structure beyond the right field near the right field line. However, prevailing southerly winds that occur during the summer months would have caused balls hit on a home run trajectory to be held up and blown back onto the playing field. To mitigate these effects, the office building was raised by two levels and a series of billboards and windscreens were added on top of the office building to "baffle" the wind as it approaches the field.

Construction Details

For the most part, connections within the structural steel frame are relatively typical, complicated only by the geometry of the structure. However, since the majority of connections were architecturally exposed, significant coordination efforts were required between the steel detailer, engineer and architect. Particular attention was focused on the wind frame connections and truss member connections, which occur in large numbers throughout the structure. To achieve the desired "old-time" look, all connections were bolted. Gusset plates first had to be sized for connection forces, then reviewed, and finally approved by the architect for relative size and configuration. Steel fabricator on the project was AISC-member Owen of Georgia, Inc., a subsidiary of Owen Steel Co. Steel detailer was MMW, Inc., and steel erector was Derr Construction Co. Design architect was David M. Schwartz/ Architectural Services, Washington, DC, and the architect of record was HKS, Inc., Dallas. General contractor was Manhattan Construction Co., Dallas. Structural engineer on the project was Walter P. Moore and Associates, Inc., Irving, TX.

Structural steel trusses were used extensively to achieve the desired architectural design. Flat-bottom trusses adjacent to curved-bottom trusses were used at each bent 32-ft.-8-in. on center to support the upper concourse. These 8-ft.-deep trusses span 30-ft. and were shop fabricated utilizing WT5 top and bottom



chords with 5 x 5 double angle web members.

Trusses also were used throughout the canopy structure above the upper seating deck. Main trusses with a depth of 3-ft.-8-in. cantilever from W36 columns located 32-ft.-8-in. on center at each seating bent. Truss chord and web members are WT6 shapes up to 60 lbs. per ft., or WT7 shapes up to 88 lbs. per ft. where lighting trusses are supported. Secondary trusses that span 32-ft.-8-in. between main trusses were fabricated using 2-1/2 x 2-1/2 double angles. Field lights are mounted within Vierendeel trusses made from 8 x 8 tube sections. These trusses are supported at the ends of the main cantilevered canopy trusses.

Steel trusses also were used in other areas by the architect. The facade of the building, which faces the playing field, consists of W12 columns and trusses fabricated from WT and double angle shapes. Similar trusses are used to express an arcade at the concession areas at the upper concourse. An attractive architectural element, the arcade also provides shade for spectators using restroom and concession facilities.

Two design concepts developed for connecting the precast seating elements to the structural steel frame provided improved sightlines and construction convenience. Traditionally, the first tread of each seating bowl is supported by a shallow structural steel member underneath. With four such conditions (two suite levels and two seating levels), floor-to-floor heights were becoming excessive. As a result, distances from seating to playing field were becoming greater, in conflict with a primary goal of the Texas Rangers to bring fans close to the field.

To minimize distances and enhance sightlines, the first tread and riser was supported by structural steel from behind the riser, not below the tread. As a result, the bottom elevation of structural steel matched the bottom elevation of the first precast tread, offering a clean architectural solution with unobstructed sightlines.

In detailing precast seating unit-to structural steel connections, the primary design goal was to provide speed and ease of erection. The solution consisted of galvanized seats bolted to the supporting bents at each riser location. Each seat had an over-sized hole in the center to receive a loose plate that had a pin projecting above and below. This loose assembly was placed on the seat in the over-sized hole, with a neoprene pad on top. The seating unit was erected onto the upper pin into a standard hole at one end of the unit to provide for thermal movement. Grout holes were detailed into the back of the galvanized seats, with the sides of the seats closed off except for small overflow holes at the corners. Once a group of units were set and final adjustments made, the seats were grouted from underneath to lock the lower pin into the seat.

Construction Sequence

The Ballpark in Arlington was constructed in a fast-track mode. Only 2-1/2 years were expended from the first day of schematic design to opening day. Construction occurred over a two-year period, leaving only six months to establish the design. Schematic design and design development lasted two months each. Construction documents were begun in January 1992. A structural concrete package was issued two months later in March 1992 that included foundations, service level, basement walls, and main concourse framing. Construction began in April 1992 and in May 1992 the superstructure was issued for bid, including all structural steel and precast seating units.

Construction of each primary element of the ballpark began behind home plate and proceeded simultaneously down the first and third baselines. Structural steel was erected using four crawler cranes, two on the playing field side and two on the outside. Erection of the 7,500 tons of structural steel occurred over a six-month period from November 1992 to April 1993, leaving exactly 12 months to complete the ballpark.

After their inaugural year in The Ballpark in Arlington, the Texas Rangers will host the All-Star game in 1995.

David A. Platten, P.E., is a vice president with Walter P. Moore and Associates, Inc., in Irving, TX.

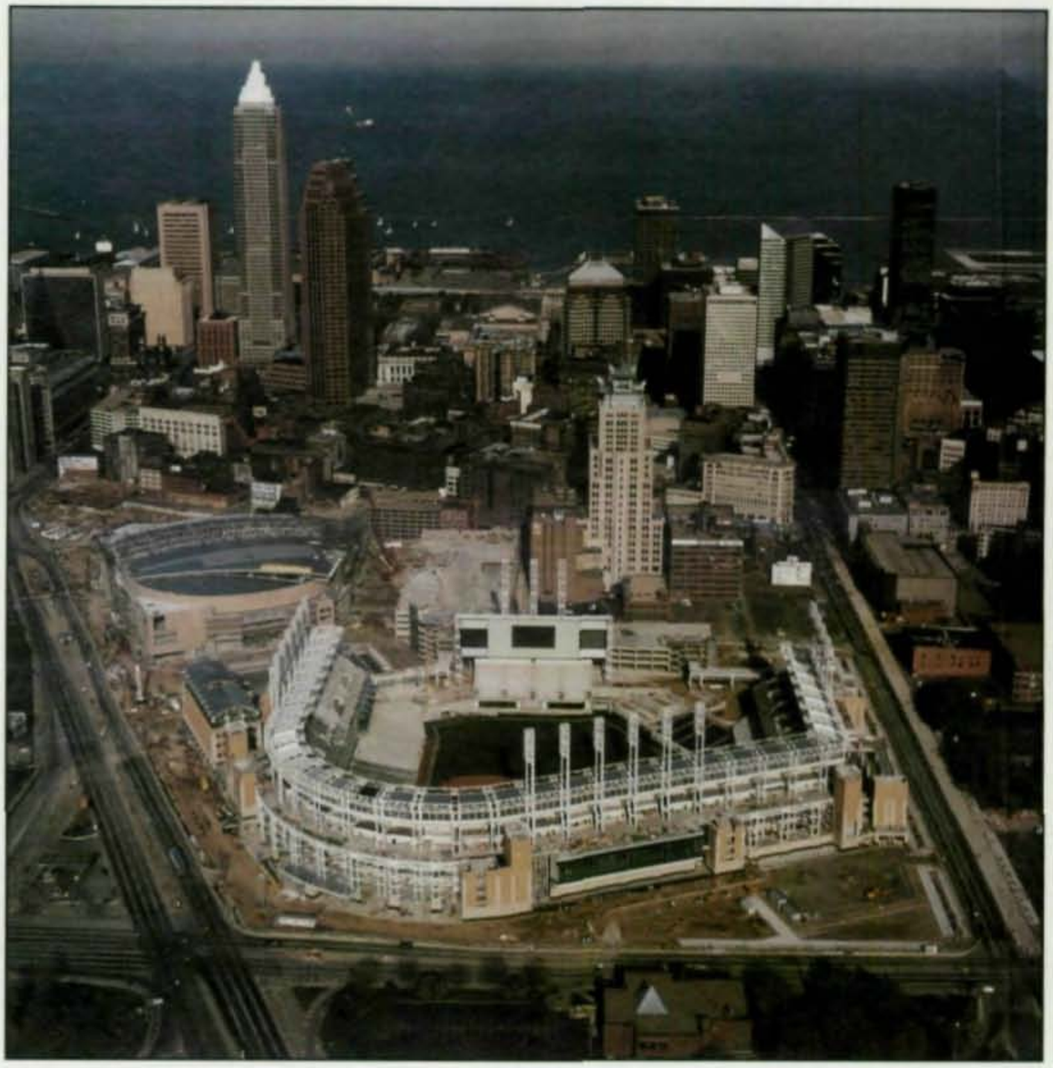
A Ballpark Without Bracing

The structural design of the Cleveland Indians Ballpark utilized trusses to visually relate the stadium to the many nearby steel bridges and mills

By Gary E. Thayer, P.E.

A steel ballpark designed without bracing? That's exactly what the structural engineers at The Osborn Engineering Company were asked to accomplish.

When the Cleveland Indians decided to replace their venerable home with a modern ballpark, they turned to HOK Sports, the architect of the Camden Yards ballpark, the very successful new home for the Baltimore Orioles. The architects suggested that a similar cross section would work for the Cleveland ballpark, but one feature needed to be eliminated: the primary vertical cross bracing system that interfered with circulation space behind the luxury suites.



Design considerations for the new home of the Cleveland Indians included aesthetic considerations, accommodating multiple levels of suites, creating unobstructed views, asymmetry, fast track scheduling, and a structural plan that all but eliminated any diagonal bracing for lateral loads.

Photos left and right by Bill Schuemann Photography.

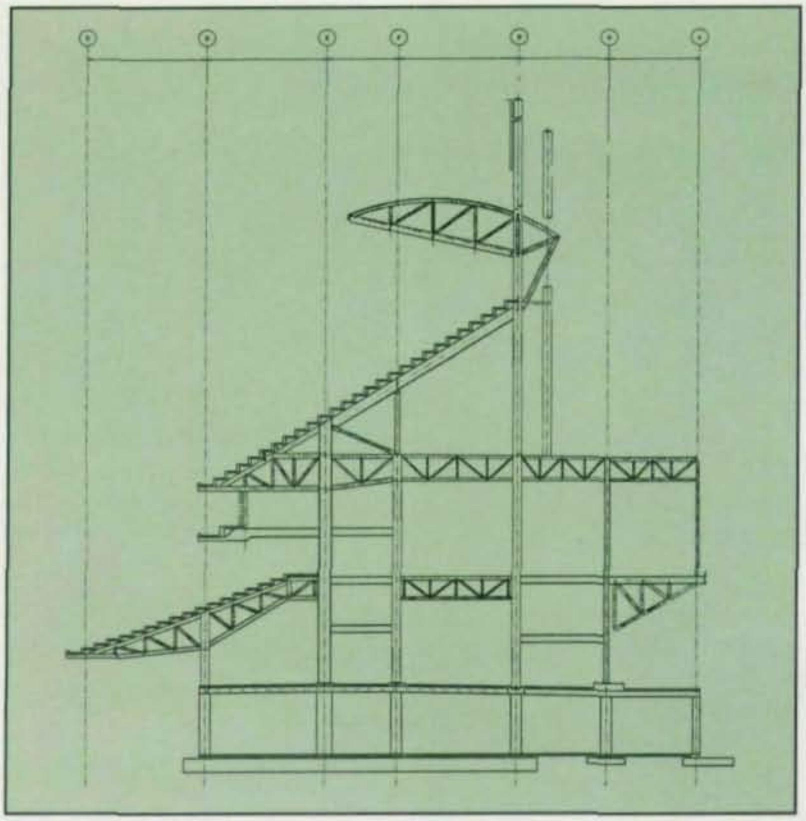
However, the vertical bracing system is the most efficient means a structural engineer has to stabilize a stadium structure and economize the structural design by enabling the use of simple connections and avoiding costly moment connections. In order to eliminate the vertical cross bracing, alternative methods of lateral support were considered and analyzed. After careful consideration, trusses were chosen in order to develop large couples within their top and bottom chords to resist lateral forces. In addition, the steel truss construction helped to visually relate the new stadium to the bridges and mill structures along the Cuyahoga River flats. Based on required spans and previous experience, trusses at least 6-ft. in depth would be required. Also, in those areas where the floor-to-floor heights wouldn't allow trusses, wide flange beams 24-in. to 36-in. in depth were used with moment connections.

Getting Started

In addition to the requirement to eliminate bracing, there were several other constraints, most of which are common to all modern ballparks and all of which played a significant role in determining the type of structure to be used. These included aesthetics, multiple levels of suites, unobstructed views, asymmetry and fast track scheduling.

For the Cleveland ballpark, a short study confirmed steel's advantages over concrete. Foremost among these were aesthetics and speed of construction. Unlike concrete, steel could be easily erected during the cold Cleveland winter.

Program requirements for the ballpark emphasized unobstructed views and three levels of suites. To meet these requirements, the structural engineer had to optimize column location versus allowable spans for cantilever framing. The design program also called for many separate "events" around the ballpark and all required a different structural response. Early in the design phase 32 separate "events", such as a stadium club and an administrative building, were recognized, and ultimately 41 separate cross sections had to be analyzed and designed. The structural response to all of these requirements was complex and needed a three-dimensional



approach to overcome the desired omission of cross bracing.

Columns

The layout of the Cleveland Indians Ballpark is very asymmetrical. Initially, the columns for this ballpark were designed as 40-in.-diameter tubes with 3-in.-thick walls and were spaced approximately 42-ft.-6-in. apart. However, given the heavy loads and moments, the original concept soon gave way to a more practical arrangement of twin 24-in.-diameter columns with $^{3}/_{4}$-in.-thick walls. The 74 columns were placed 4-ft. to each side of the 42-ft.-6-in. primary grid. The use of round columns provided the necessary three-dimensional stability for the long unbraced lengths. In addition, all of the columns in one row were staggered one half bay in plan. This created a natural triangular framing that helped make up for the lack of bracing. As a side benefit, by reducing the longest span in a typical bay by 8-ft., the need to post-tension the concrete floor at the main concourse was eliminated.

Due to the extremely large moments created by cantilevering three levels of suites 28-ft., W36x300 columns were needed at one line. With careful analysis, the cross sectional spacing of columns across a bent line was optimized to balance loads about that line and reduce the tendency for this structure to drift inward toward the playing field. The analysis of each bent line was performed using a plane frame software. This proved conservative by comparison to a three-dimensional analysis using a program from Structural Analysis, Inc., to check deflection of a typical bay. The deflection check showed potential for a drift problem at the upper concourse level and ultimately this level was anchored to concrete shear walls that were added, where possible, at stair and elevator towers for secondary forces and added redundancy.

The cross bracing that is expressed in exterior elevations does not resolve itself to the main concourse. Instead, its main purpose is to reduce the unsupported length of columns rising to support the upper concourse 60-ft. above.

Trusses And Connections

Typical trusses are composed of wide flanges with a W12 x 40 top chord, W8 x 40 bottom chord and W8 x 31 verticals and diagonals. These sizes and the panel configuration were largely chosen for aesthetic reasons.

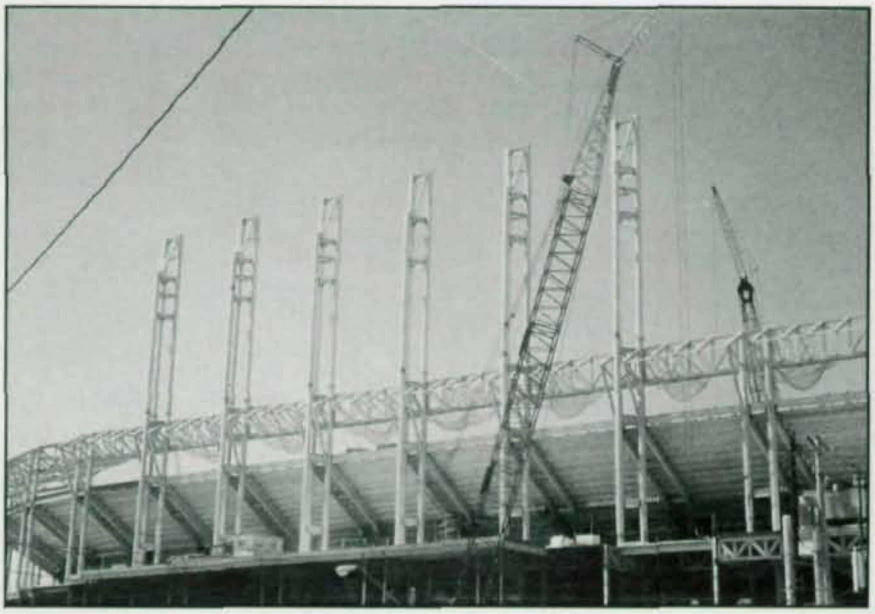
In addition to proportions, an important consideration became how to connect the trusses to the round columns while having the chords develop the required moments and axial forces necessary to develop lateral force resisting couples. While the wide flange trusses and 24-in. round columns just about eliminated the need for bracing, there were correspondingly large moments created at the connections of the trusses and girders to the columns. While $^3/_4$-in.-thick walls were all that were necessary to carry vertical loads, there was a problem anticipated from using such a relatively thin walled "shell" element to transfer the large reactions at connections of top and bottom truss chords and girder flanges. Therefore, the design team modeled the stability of the 24-in.-diameter column shell at the primary connections for local buckling by finite element methods. As expected, this analysis indicated a need to reinforce the columns at primary connections.

One proposed solution was to install internal stiffeners within the round columns, but this would have required extensive

shop labor and time and so was rejected. Another possible solution was the use of "knife" plates penetrating completely through the cylindrical column. However, that would have destroyed continuity of the cylinder and also would have been very expensive. The method finally chosen was worked out in conjunction with the project's construction manager, Huber, Hunt & Nichols, and the steel fabricator, AISC-member Kilroy Steel.

The accepted solution was to use a 24" diameter, $1^3/_4$-in. wall insert of 46 ksi steel. The 8-ft. long insert was full penetration welded to the $^3/_4$-in. wall standard pipe column and centered at the proper elevation in relation to the 6-ft. deep trusses framing to the column. In some cases, as many as five trusses were easily framed into one insert without having to add any internal stiffeners.

The exterior truss-to-column connection was complicated by the desire to have a vertical member approximately 10-in. from the face of the column. A connection plate similar in style to a long shear tab was welded continuously to this vertical and top and bottom chord extensions.



Complete penetration welds were necessary for the connection of top and bottom wide flange chords of trusses to the "insert". Slip-critical, A490 type SC bolts were used in conjunction with the full penetration welds at the top and bottom chords to resolve the high shear stresses.

The Cleveland ballpark has approximately 121 suites stacked—in most areas—three levels high. From the bottom to the top, these are called lower suite, club suite, and press suite levels. All suites are cantilevered approximately 30-ft. No columns were permitted at this area, so all three levels are suspended from the level above, and ultimately, from the cantilever trusses that also support the front portion of the upper seating bowl.

For vertical support, hangers were designed for loads up to 300 kips using two $1^1/_4$-in. x 6-in. steel plates spaced 1-in. apart at press suite level. TS 4 x4 x $^5/_{16}$ hangers were used at the lower and club suite levels. The hangers had to be on primary grid lines only and also had to fit within a metal stud wall.

So much reaction was gener-

through gusset plates into the transfer frame. The transfer frame then, by the geometry of its design, splits this reaction into two equal components half as large as the original. The connection of the floor girders to the columns were then designed for these significantly reduced loads.

Floors were designed with composite floor deck spanning over non-composite beams. The triangular configuration of framing for a typical bay with offset columns eliminated the need for studs on beams and top chords of trusses.

ated by a single line of hangers that additional diagonal struts were added at the lower and club suite levels to redistribute loads back to the primary columns. Two $1^1/_4$ x 10-in. plates were used. Because the structural framing was offset 4-ft. to each side of the hangers located on the primary grid line, the designers faced another problem: How could the large reactions in excess of 400 kips be connected back to the columns and how could the girders carrying these loads be laterally braced?

The solution to these problems was to add very stiff, triangular transfer frames made from W27 x 102 and W27 x 89 shapes within the 8-ft. space between floor girders just in front of the columns and centered on the hangers. These tie two special W27 x 102 floor girders at each level together and resolve the horizontal and vertical components of the diagonal struts

A 3-in., 20 gage composite deck with multiple spans was specified. Normal weight concrete was used and had the additional benefit of adding dead weight to counter balance the overturning moment of the three level suites. (The two pedestrian ramps and two pedestrian also were framed with this system.)

Many of the floors and levels in the ballpark are waterproofed. A wearing slab of varying thickness covers the waterproofing, which was protected with a drainage mat. The entire floor structure is sloped 2% in these areas to allow under drainage to double deck drains.

Other Considerations

The stadium club presented the most complex structural design problem associated with this project. It involved the juxtaposition of a terraced multi-level floor system at a 45 degree angle into the 42-ft.-6-in. ballpark grid system. This was further complicated by the use of suspended structural glass curtain walls to provide unobstructed vision to the ball field from the terraced dining area. Shallow, stepped trusses with heavy top and bottom chords were chosen as the dominant structural element. Additional cross bracing was needed within the depth of the truss system to laterally brace jack trusses at reaction points. Five additional columns were added below this area to support the stadium club floor framing, which is exposed and painted.

Visually, the most prominent element from all views of the ballpark are the light towers. Extending high above the sunscreen, another cantilever structural design element is used to support 19 separate banks of field lighting. Because they are arranged vertically instead of horizontally, the light towers great height required a third 24-in. round column to support their reactions and limit deflection.

As the primary support for field lighting, the towers were designed with shear tabs set at the proper location to receive prefabricated light "boxes", which were delivered to the site complete with wiring and fixtures for field lighting. The light boxes were then lifted with a crane and field bolted to the two columns extending from the primary structure.

Another large element is the world's largest free-standing scoreboard, located at the open end of the ballpark. The combined surface area, equal in area to the facade of a five-story building 200-ft. long, was structured with five triads of pipe columns with a 24-in. diameter and 1.75-in. wall. The columns are centered in an isosceles triangle with an 8-ft. base and a 7-ft. altitude. Two columns of the triad are buttressed by twin shear walls, which spring from the foundation of the center field bleachers. The connections to the shear walls via welding to $1^1/_2$-in.-thick embedded steel plates was made possible by developing the large reactions through specially detailed Lenton couplers that transferred the reactions into No. 11 rebar developed in the buttresses. In addition, three vertical banks of field lighting extend 60-ft. above the top of the scoreboard.

Additional Construction

On the northwest corner of the site, a five-story office building was constructed for use by the Cleveland Indians for administrative offices. The building is framed in steel with masonry facades.

The rectangular building was designed with shop-welded and field-bolted end-plate moment connections in both directions. Supplementary cross bracing was added at stair shafts. The building utilizes 4-in. fiber-reinforced, normal-weight concrete floors with $1^1/_2$-in. composite metal deck for floor diaphragms. The building's main architectural feature is a barrel-vaulted top, which was structured with bowstring trusses raised 16-ft. above the roof level and was supported on ornamental steel columns. The bowstring trusses are 64-ft. long and cantilever 21-ft. at each side. The top chord is a W24 x 62 curved at a 50-ft. radius.

Two pedestrian bridges constructed from trusses, columns and floor systems similar to those used on the stadium connect both ends of the club suite floor level to a 2,100-car parking garage located across the street from the stadium.

Design of a modern ballpark has become somewhat of an exercise in "structural gymnastics." In the past, cantilevers and unobstructed views played a dominant role. Now, in addition to those requirements, the Cleveland ballpark featured three suite levels and a totally asymmetrical ballpark with no vertical bracing. And, of course, the project was fast tracked with the structural design and construction being the critical path.

Despite the constraints, more than 10,000 tons of steel were designed, detailed and constructed in a period of 18 months. The structural design team had five months to finalize design and complete documents for bidding. Many details had to be worked out during the shop drawing phase and in this regard it was essential that the steel detailer have the required expertise to handle the anticipated changes and participate as a partner in the problem solving process.

Gary E. Thayer, P.E., is a project manager with Osborn Engineering Company in Cleveland.

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Chicago, IL 60601-2001

FAX 312/670-5403

Computer Software For Fabricators And Detailers

The following is a listing of companies that supply software for detailing, estimating, inventory, and production. Included is information about the software, what makes it different than its competitors and cost.

Company Name: **AISC**
Address: One East Wacker Dr., Suite 3100
Chicago, IL 60601-2001
Phone Number: (312) 670-2400
Fax Number: 312/670-5403
Types of Software: 1) Steel shape generator for AutoCAD; 2) Connection Design; and 3)Beam web opening design
Major Features: 1) AISC for AutoCAD will draw the end, elevation, and plans of W, S, M and HP shapes, American Standard Channels (C), Miscellaneous Channels (MC), Structural Tees cut from W, M, and S shapes (WT, MT, ST), Single and Double Angles, Structural Tubing, and Pipe. Shapes are drawn to full scale and correspond to data published in Part 1 of the *AISC Manuals of Steel Construction*. U.S. or Metric units may be selected.
2) Also available is CONXPRT, a program for the complete design of shear and moment connectins and column stiffeners and doublers. Both ASD and LRFD versions are available.
3) WEBOPEN analyzes and designs reinforcement (if required) for steel and composite beams with circular or rectangular web openings.
Unique Aspects: 1) AISC for AutoCAD is the only AutoCAD shape generator sponsored by AISC.
2) CONXPRT is based on the AISC Manual of Steel Construction and Volume II—Connections. It combines the engineering knowledge and experience of respected fabricators and design engineers. The menu-driven program, complete with built-in shapes, provides complete documentation of all design checks.
3) WEBOPEN is the only commercially available program for this function.
Cost: 1) AISC for AutoCAD—$120
2) CONXPRT —$110 - $820
3) WEBOPEN — $495
Free Demo: Only for CONXPRT.

Company Name: **AutoSD, Inc.**
Address: 4033 59th Place
Meridien, MS 39307
Phone Number: (601) 693-4729
Fax Number: 601/693-4729
Type of Software: Structural & miscellaneous detailing inside AutoCAD
Major Features: Details beams, columns, braces, gusset plates, stairs, stair rails and wall rails. It also has several programs and tools for creating erection drawings. The user also can automatically draw at different scales in the same drawing.
Unique Aspects: Data base of AISC shapes allows the program to calculate detailing dimensions. Stair program details pan & grating trade stairs. Stairs can be detailed sloping in either direction. and user can create own pan design to be used by stair program. The gusset plate program completely designs the brace connection and will draw the plate to scale. Information from the gusset is used to draw the brace.
Cost: $3,500.
Free Demo: Yes.

Company Name: **Barry R. Bowen Associates**
Address: 3394 Coleman Road
Memphis, TN 38128
Phone Number: (901) 373-6468
Fax Number: 901/373-6468
Type of Software: AISC structural shapes within AutoCAD
Major Features: Draws all of the AISC structural shapes in plan, section, elevation, and single line. For AutoCAD R10-12, with both DOS and Windows versions. Selection is made from a dialog box that includes the shape properties and icon for view selection.
Unique Aspects: Each shape is controlled in its own unique dialog box for easy selection. Shapes are created parametrically the first time using a polyline and made into a block. Layers and hatching are created automatically. Properties of shapes within drawings may be checked at any time after insertion.
Cost: $89.97 (includes free bonus module)
Free Demo: No.

Company Name: **Computer Detailing Systems, Inc. (CDS)**
Address: 7280 Pepperdam Ave.
Charleston, SC 29418
Phone Number: (803) 552-7055
Fax Number: 803/552-3455
Type of Software: Structural steel detailing
Major Features: Produces AutoCAD compatible drawings of shop details of beams, columns, vertical bracing, horizontal bracing, stairs, girts, purlins, lintels, trusses, bents, and plate girders. Drawings are complete with dimensions, bill of materials and welds shown. Produces connec-

tion engineering calculations, CNC data, and production control data. Full capability to interface with design firms through the steel detailing neutral file including 3D modeling, automated detailing and advanced mill ordering.

Unique Aspects: Versatility (uses the fabricator's standards), flexibility (user does not have to input an erection drawing; therefore revisions, "holds", etc. are handled with ease); and adaptability (ability to customize connections that may be unique to a specific job).

Cost: $15,000 - $25,000

Free Demo: Yes.

Company Name: ***Cad*Vantage, Inc.**

Address: 619 South Cedar St., Studio A, Charlotte, NC 28202

Phone Number: (704) 344-9644

Fax Number: 704/358-1801

Type of Software: Structural steel detailing

Major Features: *Cad*Vantage Structural Version 5.5 (CSV) is a completely automated, stand-alone detailing system that combines the flexibility of individual input with the speed of batch processing. Users have total control of the detailing process. They can allow CSV to perform all tasks, or take control of any aspect on a global or individual basis for unusual conditions, which allows the program to be used for any job. Also, CSV allows users to to add, edit or create new system connections of any type or configuration to the CSV Connections Library. Pertinent information regarding material requirements is automatically assembled (from the Advanced Material List Program or on a per sheet basis) for direct import into external production software systems, which reduces redundant keying of data.

Unique Aspects: The program is very easy to use, has excellent documentation, a context sensitive on-line help system, and offers experienced technical support. The company plans on introducing a new Windows graphical interface system that runs on pen computers (and recognizes handwriting) at the National Steel Construction Conference in May. This new media makes full use of reusable, user defined icons that control everything from entire end or midspan connections to column base and cap definitions.

Cost: $8,995 (monthly leases are available).

Free Demo: Yes, including a self-running tutorial.

Computer Software For Fabricators And Detailers

Company Name: **Computer Detailing Corporation**
Address: 80 Second Street Pike #10
Southampton, PA 18966
Phone Number: (215) 355-6003
Fax Number: 215/355-6210
Types of Software: Framing Plans, Detailing, & Multing
Major Features: Computer Detailing's Plans & Elevations software is a menu driven program that works inside of AutoCAD. Various scales can be used in the same drawing, without any calculations or conversions and structural shapes and elements can be automatically drawn to scale or exaggerated. In addition, roof frames, railings, girts and other building elements can be drawn automatically and space frames, trusses, stair plans and miscellaneous items can be detailed. The program is designed for use by structural steel and miscellaneous fabricators.
The company's Beams & Columns program can create details for any structural element or fitting. The user-friendly program does not require extensive training and was designed for use by detailers with no previous computer experience. The program works within AutoCAD and finished details are visible as they are created. Shop cutting lists can be created and a separate metric version is available. Also available is a separate module for detailing stair stringers.
The company's Optimal Cutting Program will find the most economical nesting of material from either inventory or warehouse stock lengths or a combination of both. It uses a sophisticated algorithm that tries thousands of combinations.
Unique Aspects: Beams & Columns offers input from a descriptive menu or from system prompts. Users can create their own standard marking system as well as customize their own sheet size, format and bill of material. Free unlimited support is available and their is no annual license or upgrade fee.
Cost: Plans & Elevations starts at $2,200.
Beams & Columns starts at $4,850.
Optimal Cutting Program is currently offering a special introductory price of $129
Free Demo: Call for info.

Company Name: **Design Data**
Address: 121 South 13th St., Suite 204
Lincoln, NE 68508
Phone Number: (402) 476-8278
Fax Number: 402/476-8354
Type of Software: Engineering, Detailing, Production, Estimating Design & CNC
Major Features: The SDS/2 Steel Fabrication System is the only

integrated system for steel fabricators. The Detailing Module is the central hub of information. Not only does it increase productivity in drafting, it creates information that allows managers to improve their fabrication efficiency.

Unique Aspects: During the past 12 years, Design Data modules have improved in capability and efficiency. Design Data's products automate all types of steel fabrication. From structural to miscellaneous steel design, the system automates each fabricators unique system.

Cost: Call for pricing.

Free Demo: In-person demonstrations available.

Company Name: **E.J.E. Industries, Inc.**

Address: 287 Dewey Ave.
Washington, PA 15301

Phone Number: (412) 228-8841 or (800) 321-3955

Fax Number: 412/228-7668

Type of Software: Estimating and Production

Major Features: For estimating, the Structural Material Manager accepts entries for material cost, shop hours and field hours and produces a job summary based on this information. Total item weights are provided via built-in steel, stainless and aluminum weight libraries. In addition, total paint and primer costs are provided. Once the job is in production, the material nesting feature can be used to find the optimal cut of items from available stocks. As fabricated items are shipped, a Production Control Module prints shipping tickets and automatically records the date and quantity shipped for each piece mark. An inventory function maintains the user's list of in-house stock. And a newly added Plate-Nesting Module finds the optimum cut of square and rectangular items from stock plates.

Unique Aspects: The Nesting Module produces extremely efficient cut lists by running thousands of combinations on the items. The system can accept dimensions in both metric and U.S.

Cost: $1,000 - $2,000

Free Demo: Yes. Includes full system's operator's manual.

Company Name: **NES**

Address: P.O. Box 2014
El Segundo, CA 90245

Phone Number: (800) 637-1677

Fax Number: 310/541-6738

Type of Software: Calculations for steel detailing

Computer Software For Fabricators And Detailers

Major Features: The program: creates bolt lists & bolt summaries; calculates the camber of a beam or truss; solves right and oblique triangles, circles and arcs; designs gusset plates; designs connections in tension & shear; views dimension properties of steel shapes; designs beam connections using clips, shear end plates, seats or wing plates.

Unique Aspects: This calculator program is a PC solution that provides 11 different modules for calculating the most common steel detailing and checking problems. A main menu links all modules, providing for easy calculation selection.

Cost: $250

Free Demo: 30-day money back guarantee.

Company Name: **Romac Computer Services, Inc.**

Address: 332 S. Main St., P.O. Box 660
Lake City, TN 37769

Phone Number: (615) 426-9634

Fax Number: 615/426-6454

Type of Software: Production, Purchase Orders, Length Nesting Plate Nesting & Detail Drawing Log

Major Features: Written specifically for the steel fabrication industry, the software has undergone numerous enhancements and additions since first introduced in 1982. The Fabrication Package can be purchased as individual modules or as an integrated system. Features include: material management; shop cutting lists; production tracking; shipping tickets; and other fabrication shop management tools.

Unique Aspects: The software is easy to use and implement. Free telephone support is available.

Cost: $295 - $1,295

Free Demo: Yes for most modules. A 30-day trial with satisfaction guaranteed offered for all modules.

Company Name: **SSDCP**

Address: 110 Shady Oak Circle,
Florence, MS 39073

Phone Number: (601) 845-2146

Fax Number: 601/845-2146

Type of Software: AutoCAD parametric LISP programs.

Major Features: SSDCP offers more than 140 LISP programs that run inside of AutoCAD Release 10 and up for the creation of shop drawings of structural and miscellaneous steel. These are not block or library programs and instead offer everything from anchor bolt plans to roof opening frames—and everything in between. All details are drawn on the monitor while you watch, so you stay in control of the detailing at all times.

Unique Aspects: A full 30-day, money-back guarantee (excluding shipping costs) is offered. The program runs on your existing hardware (IBM compatible). It's been used in the field for more than 8 years. There's no maintenance fee; one update is provided annually for a nominal fee. Each program can be purchased separately. Free phone support is offered.
Cost: Starts at $395.
Free Demo: Yes.

Company Name: **Softdesk, Inc.**
Address: 7 Liberty Hill Road
Hennicker, NH 03242
Phone Number: (603) 428-3199
Fax Number: 603/428-7901
Type of Software: Steel detailing.
Major Features: This detailing program works with R12 AutoCAD on either DOS or Windows. U.S, Canadien and European shapes are provided. All structural data is stored in user customizable DBF format. The program automatically formats a Bill of Materials and calculates weights. Special utilities are provided for drawing beams, columns, bracing, stairs, etc.
Unique Aspects: Softdesk offers a Windows version and is AutoCAD-based. The use of a DBF file format makes is easily customizable.
Cost: $2,995
Free Demo: No.

Company Name: **Steel Solutions Inc.**
Address: Route 3, Box 312A
Buckhannon, WV 26201
Phone Number: (304) 472-2668
Fax Number: 304/472-3214
Type of Software: Structural fabrication management systems; packages include Fabricator, Estimating, Service Center, Purchasing, Inventory Control, Drawing Control & Accounting.
Major Features: Steel 2000 features include: pull-down menus; context sensitive help; full mouse support; sleek graphic interface; quick execution speed; substantial browse and change capabilities; and the ability for the end user to create and maintain their own custom records.
Unique Aspects: Steel 2000 is a totally integrated solution to steel fabrication management. It has been and continues to be developed at a state-of-the-art fabrication facility, Steel Service Corporation, in Jackson, MS. It is written in Foxpro, an advanced relational database management system for microcomputers. The program is furnished with complete technical documentation covering the data structures, reports, indexes and relation keys. The system's open architecture design provides access to all data within the system. Reports may be customized for any imaginable purpose. The program's CNC capabilities allow the production activity of all automated equipment to be monitored in real time as production occurs and a record for each CNC machine is kept for pieces produced for a work shift, average minutes required for each piece, minutes since the last piece was produced, and the current project and cutting list.
Cost: Starts at $10,000.
Free Demo: Yes. Sample reports, brochures and a video also are available upon request.

Company Name: **Structural Software Co.**
Address: 5012 Plantation Road, N.E.
Roanoke, VA 24019
Phone Number: (703) 362-9118
Fax Number: 703/366-6036
Type of Software: Estimating, production control, inventory, purchase orders and combining.
Major Features: The software is available in interactive modules so the users can build a system to match their individual needs. Estimating offers a complete solution for automating an estimating department. The system calculates weights, surface area, material cost and labor, and will even count shop and field bolts. Production Control gets jobs into the shop by providing cutting lists and then ushers it through, all the way to shipping. The job may be released into the shop by sequence, drawing number, category, main piece or accessory piece and is then tracked from station to station. Inventory Control provides fingertip access to normal stock as well as drops left over from previous jobs. Purchase Orders automatically integrates the Inventory Control module with the purchasing department. Combining optimizes the cutting of material and can be interfaced with all of the other modules. And finally, the Automated Drawing Log is designed to track the vast number of drawings needed for each job.
Unique Aspects: The company's modules are used daily by more than 450 fabricators nationwide. SSC has been providing products and services to the steel industry for more than 10 years. The programs can be purchased individually for a specific function or in groups to form an interactive system. The company offers complete support staff for customer service and training and the software is normally upgraded twice a year with all customers covered by software maintenance automatically receiving these upgrades.
Cost: $299 to $5,000.
Free Demo: Yes.

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P.O. Box 81096, Lincoln, NE 68501

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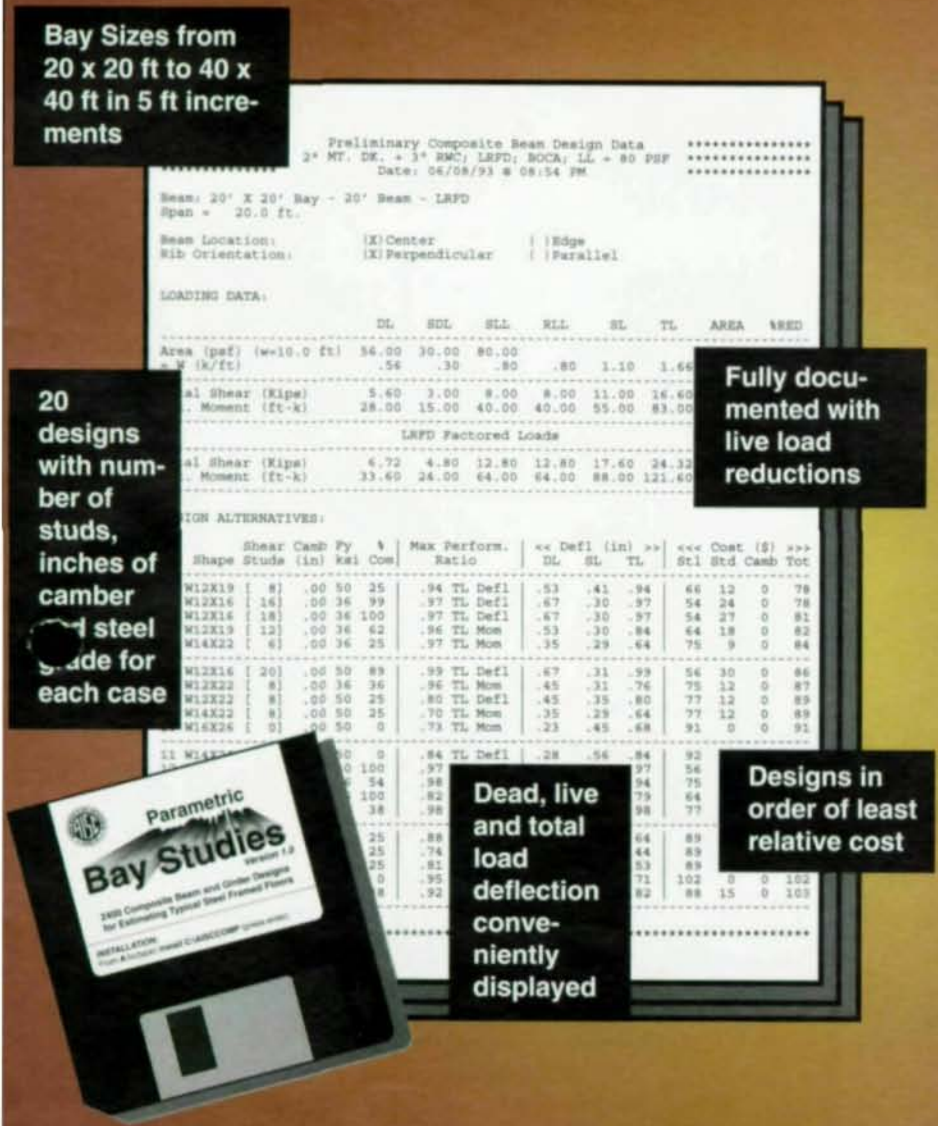

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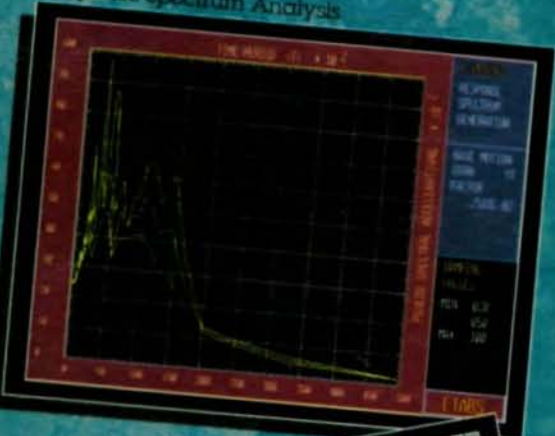

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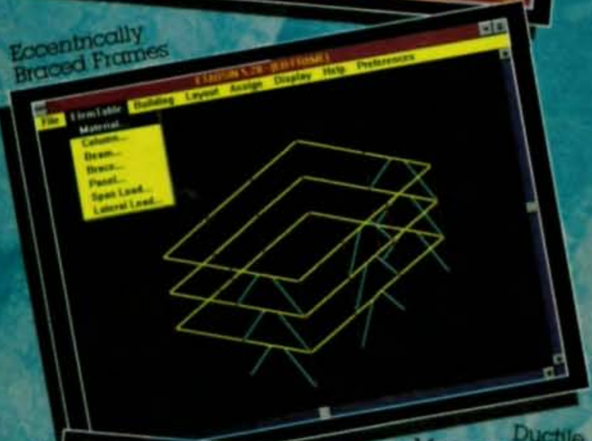

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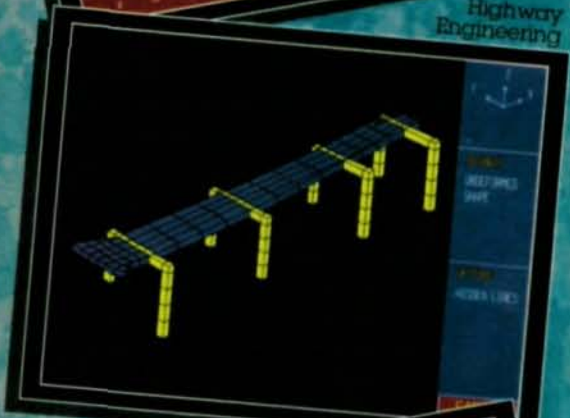

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