MODERN STEEL CONSTRUCTION

May 1995

$3.00

National Steel Construction Conference

Structural Engineers/Managers

Uplift Values for SCREWED DECK

Deck can be screwed to structural steel, bar joists, or light gage steel framing. The lowest strength was used to produce the tabulated values. For bar joists and structural steel, a tensile strength (F_u) of 58 ksi was used which is the lowest value for A36 steel. For gage supports, F_u = 45 ksi was used which is the lowest provided in ASTM A653 Structural Quality grade 33. Deck materials furnished in gages 24, 26 and 28 are usually grade 80 steels which use a tensile strength (F_u) of 60 ksi as limited by the AISI specifications. Either pull out of the screw or pullover of the deck will normally control. The values are based on the equations provided by the AISI Specifications (1986 with addenda). These specifications call for a safety factor of 3 to be applied to the table values. However, for temporary wind loads, a one third load increase is appropriate. If it is known that the tensile strength of the support steel or the sheet steel is greater than the values used for the tables, the tabulated ultimate strengths may be increased by a straight line ratio.

Screw Size	d dia.	d_w nom. head dia.	Average tested tensile strength, kips
#10	0.190	0.415 or 0.400	2.56
#12	0.210	0.430 or 0.400	3.62
1/4	0.250	0.480 or 0.520	4.81

Pull Over Strength, kips = $P_{nov} = 1.5\, t_1 d_w F_u$; $d_w < 0.50"$.

Washer or head, d_w	16 ga.	18 ga.	20 ga.	22 ga.	24 ga.	26 ga.	28 ga.
0.400	1.61	1.28	0.97	0.80	0.86	0.64	0.54
0.415	1.68	1.33	1.00	0.83	0.89	0.67	0.56
0.430	1.74	1.38	1.04	0.86	0.92	0.69	0.58
0.480	1.94	1.54	1.16	0.96	1.03	0.77	0.64
0.520(0.500)	2.02	1.60	1.20	1.00	1.08	0.81	0.67

Key
Fu = 60 ksi
Fu = 45 ksi
Fu = 58 ksi

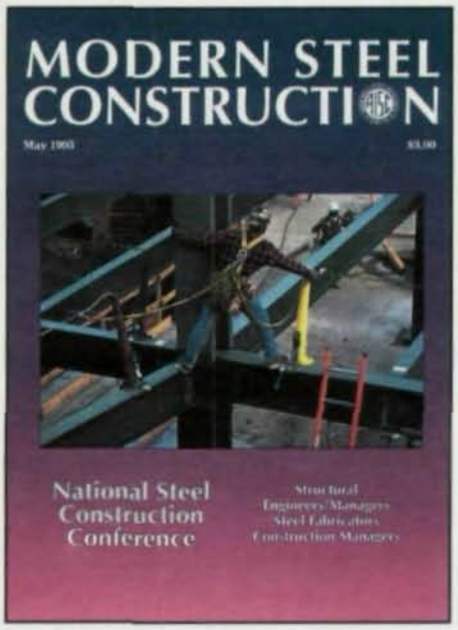
Pull Out Strength, kips = $P_{not} = 0.85\, t_2 d F_u$; Metal thickness = t_2

Screw	1/4"	3/16"	10 ga. (0.135)	1/8"	12 ga. (0.105)	14 ga. (0.075)	16 ga. (0.060)	18 ga. (0.047)	20 ga. (0.036)	22 ga (0.030)
#10	2.34	1.76	0.98	1.17	0.76	0.55	0.44	0.35	0.26	0.22
#12	2.66	2.00	1.12	1.33	0.87	0.62	0.50	0.38	0.29	0.24
1/4	3.08	2.31	1.29	1.54	1.00	0.72	0.57	0.45	0.34	0.28

Note: In our Metric catalog "Steel Decks for Floors and Roofs", the tables on pages 33 and 35 are in the wrong place. Contact us for the needed corrections or contact us for a copy of the corrected publication.

MODERN STEEL CONSTRUCTION

Volume 35, Number 5 May 1995



This year's National Steel Construction Conference features a wide range of seminars on such diverse topics as "seismic design" and "limiting a firm's liability." A Reader's Digest *view of some of some of the conference papers begins on page 16.*

Modern Steel Construction (Volume 35, Number 5). ISSN 0026-8445. Published monthly by the American Institute of Steel Construction, Inc., (AISC), One East Wacker Dr., Suite 3100, Chicago, IL 60601-2001.

Advertising office: The Ramage Group, O'Hare Lake Office Plaza, 2400 E. Devon Ave., Des Plaines, IL 60618 (708) 699-6049.

Subscription price:
Within the U.S.—single issues $3; 3 years $85.
Outside the U.S.—single issues $5; 1 year $36; 3 years $100.

Please fax any addresses changes to 312/670-5403 *or to the address below.*

Postmaster: Please send address changes to Modern Steel Construction, One East Wacker Dr., Suite 3100, Chicago, IL 60601-2001.

Second class postage paid at Chicago, IL and at additional mailing offices.

FEATURES

1995 NATIONAL STEEL CONSTRUCTION CONFERENCE

DEPARTMENTS

Check The Mill Reports

IF YOU'RE AN ENGINEER WHO REGULARLY DESIGNS WITH A36 STEEL, you should be aware that not all Grade 36 steels are created equal. The technology used today to produce structural steel shapes is developing strengths similar to that found in A572 Grade 50, even in A36 steels. Due to these yield strengths, the industry is moving toward a new steel standard for building design that will have a 50 ksi yield rather today's 36 ksi yield.

While this may be somewhat confusing, help is on the way. AISC's 1995 lecture series, "Advances In Structural Steel", will discuss the new 50 ksi yield strength steel designed to replace ASTM A36 as the industry base standard. The seminar will answer engineer's questions about this new material's effect on design and construction, including minimum and maximum material strength, ductility and economic considerations.

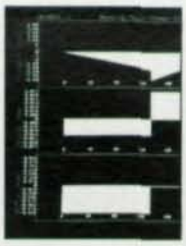
In addition, the seminar will provide practical information on designing in LRFD. Yes, I know a lot of you are probably tired of reading about how wonderful LRFD is. But regardless of how hesitant you are to switch from ASD, LRFD is the preferred steel specification. This seminar is designed to present straightforward, basic procedures for designing members and connections in LRFD.

Perhaps the best part of the seminar, however, will be a presentation on the most commonly asked questions to AISC's staff engineers. There'll be lots of good information on bolting, bracing, coatings, and connection design in general. Discussion will be encouraged, so show up with some questions in mind.

Finally, the seminar will include the latest information gleaned from research after the Northridge Earthquake. Seismic design is not for the west coast only; the Northeast has strict seismic guidelines and much of the midwest sits all-to-close to the New Madrid fault.

For information on when the seminar series will be in your area, see pages 12-13. **SM**

How to get from here



Engineering, Analysis and Design Module

Estimating Module

Detailing Module

Production Control Module

CNC Interface Module

to here.

Design Data's SDS/2 Steel Fabrication System.

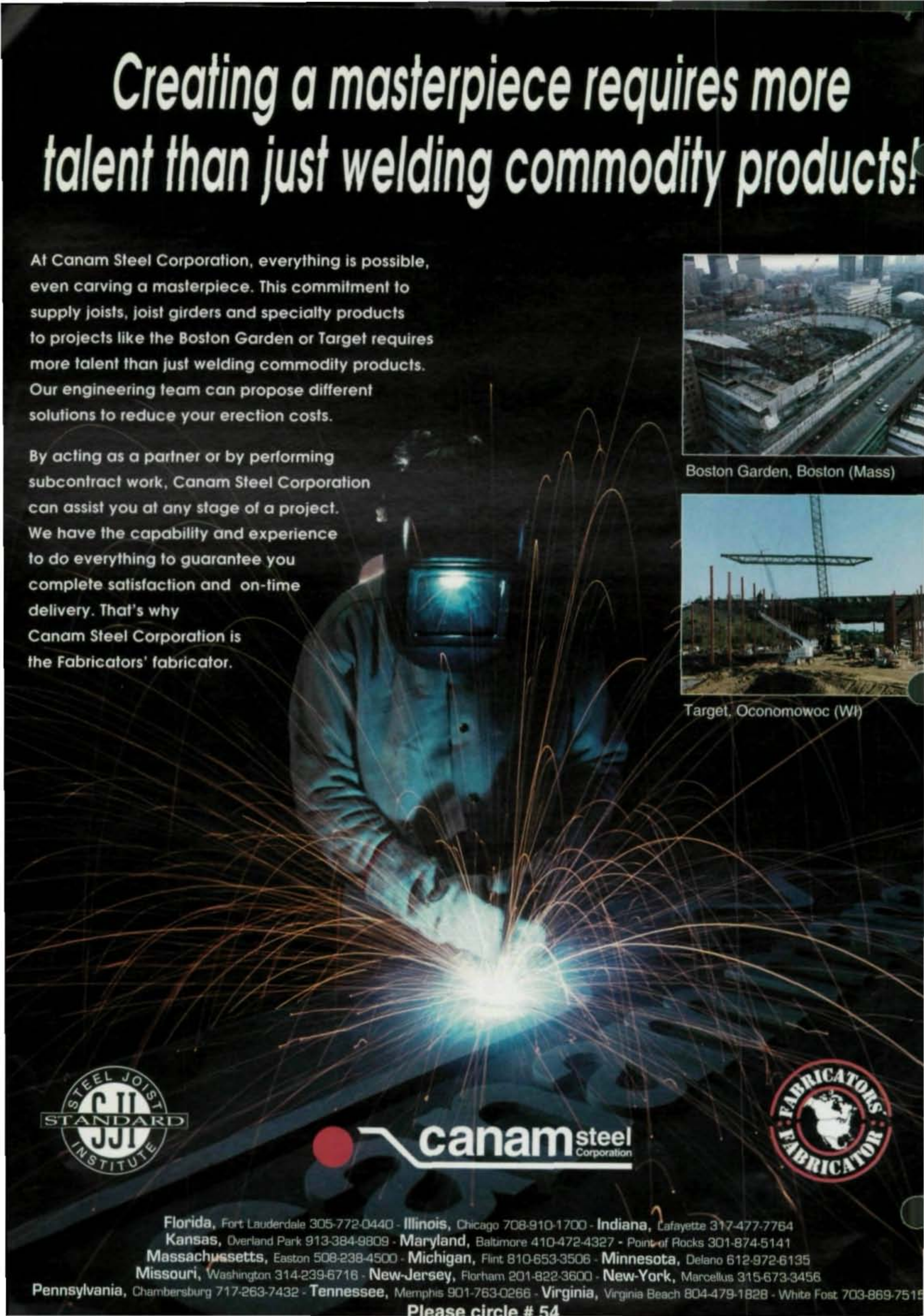
SDS/2 gives you the flexibility to integrate all aspects of your business with one software system. That concept is called Information Management. Each module by itself will save you time and money and by combining products to implement Information Management you receive more than twice the benefit in savings and productivity. So whether you need one SDS/2 software module or all these tools working together, Design Data can provide the most productive system for you.

For more information about SDS/2, information management in the steel industry or future product demonstrations call **800-443-0782**.

Please circle # 32

DESIGN DATA

"First in...software, solutions, service"
402-476-8278 or 1-800-443-0782



Creating a masterpiece requires more talent than just welding commodity products!
At Canam Steel Corporation, everything is possible, even carving a masterpiece. This commitment to supply joists, joist girders and specialty products to projects like the Boston Garden or Target requires more talent than just welding commodity products. Our engineering team can propose different solutions to reduce your erection costs.
By acting as a partner or by performing subcontract work, Canam Steel Corporation can assist you at any stage of a project. We have the capability and experience to do everything to guarantee you complete satisfaction and on-time delivery. That's why Canam Steel Corporation is the Fabricators' fabricator.
Boston Garden, Boston (Mass)
Target, Oconomowoc (WI)
STEEL JOIST INSTITUTE
SJI
STANDARD
canam steel Corporation
FABRICATORS' FABRICATOR
Florida, Fort Lauderdale 305-772-0440 - Illinois, Chicago 708-910-1700 - Indiana, Lafayette 317-477-7764
Kansas, Overland Park 913-384-9809 - Maryland, Baltimore 410-472-4327 - Point-of Rocks 301-874-5141
Massachussetts, Easton 508-238-4500 - Michigan, Flint 810-653-3506 - Minnesota, Delano 612-972-6135
Missouri, Washington 314-239-6716 - New-Jersey, Florham 201-822-3600 - New-York, Marcellus 315-673-3456
Pennsylvania, Chambersburg 717-263-7432 - Tennessee, Memphis 901-763-0266 - Virginia, Virginia Beach 804-479-1828 - White Fost 703-869-7515
Please circle # 54

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

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

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

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

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

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

Are there any limitations on the span to depth ratio of beams required by AISC Specification for Structural Steel Buildings?

One guideline that is given by AISC can be found in the Allowable Stress Design Specification Commentary for Section L3.1, which deals with serviceability design considerations. The commentary states the following:

"Although deflection, rather than stress, is sometimes the criterion of satisfactory designs, there is no single scale by which the limit of tolerable deflection can be defined. Where limitations on flexibility are desirable, they are often dictated by the nature of collateral building components, such as plastered walls and ceilings, rather than by considerations of human comfort and safety. The admissible amount of movement varies with the type of component. The most satisfactory solution must rest upon the sound judgement of qualified engineers. As a guide, the following rules are suggested:

1. The depth of fully stressed beams and girders in floors should, if practicable, be not less than (F_y/800) times the span. If members of less depth are used, the unit stress in bending should be decreased in the same ratio as the depth is decreased from that recommended above.

2. The depth of fully stressed roof purlins should, if practicable, be not less than (F_y/1000) times the span, except in the case of flat roofs."

Although these are only suggested guidelines and not strict limitations, they offer some useful assistance to the question.

Mark D. Hartle, P.E.
Pruitt Eberly Stone, Inc.
Atlanta, GA

What criteria is used to design "NOT STRUCTURAL STEEL" members such as stairs, catwalks, handrail, and toeplates?

For those "NOT STRUCTURAL STEEL" member design concerns, the designer can use either the certified engineering data and material specifications from the individual metal manufacturer/supplier or the Metal Stairs Manual, Pipe Railing Manual, Catwalks Manual, and so on, published by the National Association of Architectural Metal Manufacturers (NAAMM) in addition to the local building codes.

As regard to the load criteria,the building code shall cover the minimum design loads for each subject item, for example 1991 Uniform Building Code, Table 23-A and 23-B.

Kunming Gwo, P.E.
HCI Steel Building Systems, Inc.
Arlington, WA

The *Manual of Steel Construction* includes many items that are used along with structural steel frames, this is very convenient for structural engineers. However, some of the tables do not provide all of the information needed by engineers. One of the tables that AISC includes covers the dimensioning of cotter pins. What is the strength of cotter pins listed in the Manual of Steel Construction? Where can these items be obtained?

Cotter pins are commonly constructed of type AISI 1010 low carbon steel or type 302 stainless steel. Dimensions of the pins and the recommended hole size are covered by ANSI Standard B18.8.1. Dimensions are also tabulated in various machine design books such as *Machinery's Handbook*. Knowing the dimensions and the material (low carbon steel or stainless steel) one can calculate the shear strength of cotter pins. Cotter pins are manufactured and sold by numerous companies. Local distributors are usually found in the Yellow pages under "Fasteners" or "Bolts and Nuts". Usually distributors of slotted nuts, clevises and similar hardware sell cotter pins even though their Yellow page advertisement may not say so. Manufacturers are listed in the *Thomas Register* under "Cotter Pins" and "Pins: Cotter". *Machinery's Handbook* and the *Thomas Register* can be found in many public or college libraries.

Doug Werner
Douglas Engineering
Westminster, CO

In a partially cover-plated column, how would you analyze the column for governing l/r ratio to calculate F_a?

In the 1st Quarter 1979 AISC *Engineering Journal*, an article with an approximate method is put forward dealing with this question. It requires the calculation of the Euler buckling load of the entire column by some process, typically numerical, and the identification of the most highly stressed segment due to the axial load. Although the column will buckle as a whole, the most highly stressed segment can be used to find an effective Kl/r leading to an allowable axial load for the entire column. Allowable stresses for all segments will follow.

C. P. Mangelsdorf
University of Pittsburgh
Pittsburgh, PA

New Questions

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

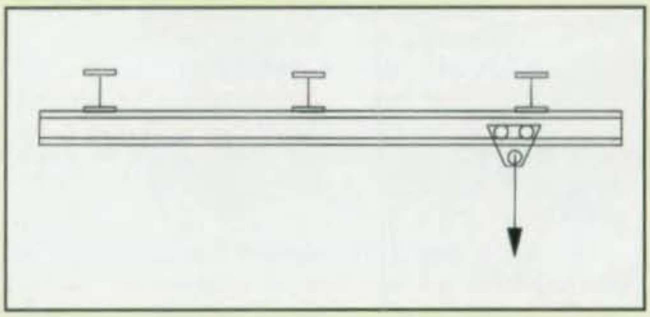
If you have an answer or suggestion please send it to the Steel Interchange Editor, Modern Steel Co nstruction, One East Wacker Dr., Suite 3100, Chic ago, IL 60601-2001.

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

For a continuous trolley beam with multiple spans and cantilevered ends what is the lateral unbraced length for the bottom flange?

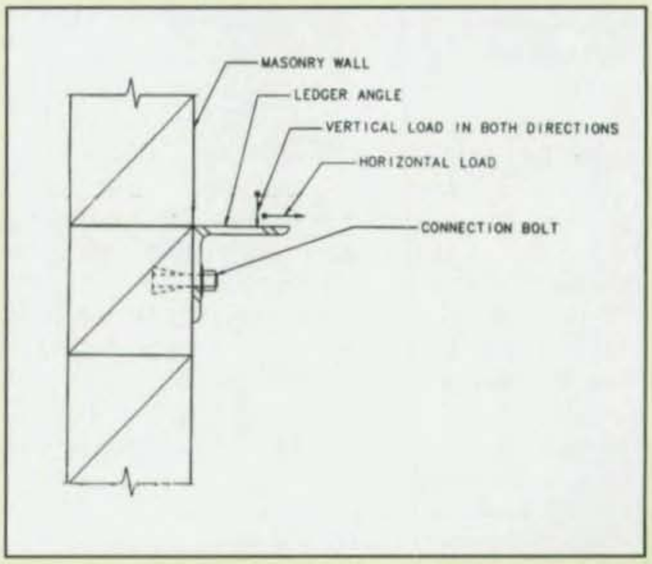
Can the distance between points of moment inflection be considered an unbraced length?

Larry Nix, P.E.
Dallas, TX



What information is available to provide guidance in the design of a ledger angle and its connection to a wall in particular, with the bolt in tension and the lower edge of the vertical leg of the angle in compressive contact with the wall, what is the stress distribution in the angle? Is only part of the angle effective in resisting the applied loads, based on the spacing of the bolts?

Greg Michel, P.E.
Mani Muthiah, M.S.E.
Lescher & Mahoney/DLR Group
Phoenix, AZ



IF you think ZINC is only a vitamin supplement, THINK AGAIN

Much more ZINC is used to protect steel structures against corrosion.

"Today" HOT DIP GALVANIZING is a proven, cost effective reliable application to provide cathodic protection for decades on structural steel, fasteners, rail, material handling, bridges, scaffolding...

Always a step ahead, we installed the first 8'6" deep galvanizing kettle in the United States. This allows us to produce a high quality product, and to galvanize fabricated structures up to 16' high by 45' long.

What about a hot dip galvanized 1/4" bolt. We can do it because we operate the only "High Temperature" kettle for centrifuge work.

Count on our experience with painting over galvanizing. We offer the one source "Duplex System".
Our freight service operates company-owned trucks and trailers, and if needed, will arrange shipping with reliable haulers.

With over 100 years experience, 4 U.S. locations, and over 30 plants worldwide, the group of Voigt & Schweitzer is your BEST source for hot dip galvanizing.
Voigt & Schweitzer means experience. We like sharing our experience with you, and are presenting educational seminars to professionals and fabricators on hot dip galvanizing.

Columbus Galvanizing
Voigt & Schweitzer, Inc.
Columbus, OH
(614) 443-4621

Voigt & Schweitzer
Galvanizers, Inc.
Redford, MI
(313) 535-2600

V&S Pilot Galvanizing
Poca, WV
(304) 755-2949

V&S Progalv
Muskogee, OK
(918) 683-5005

Member American Galvanizers Association, CSI, AIA, AISC, ASA. ASTM Spec/Military Spec 45208A qualified.

We designed our mill with the same convenience in mind.
When you're in the market for steel, and you need it fast, you know there's nothing convenient about waiting for a rolling schedule. Maybe it's time you switched from a rolling mill to a stocking mill: Chaparral.
Our $50-million on-site steel inventory is one of the largest in the world. It's so large, in fact, that we can fill over 80% of our steel orders from stock — in two weeks or less. Special orders? Fast track jobs? We'll process those within 72 hours. Even if your order changes, we'll work with you to fill it as quickly as possible.
So what are you waiting for? Next time you're shopping for steel, call Chaparral. You'll always get the right steel…right away.
CHAPARRAL STEEL
Toll Free (800) 527-7979 U.S. and Canada • Local (214) 775-8241
Fax (214) 775-6120 • 300 Ward Road, Midlothian, Texas 76065-9651
Please circle # 43

FOCUSING ON PRACTICAL STEEL DESIGN

FROM DISCUSSIONS OF BOLT INSTALLATION TO THE DEVELOPMENT OF A NEW HIGH-STRENGTH STEEL, AISC's 1995 Seminar Series is designed to provide practical information for structural engineers, fabricators, and others involved in the steel construction industry.

"Fast moving developments in structural steel may have been difficult to absorb in the past, but now events are focusing and clarifying the issues," according to Robert F. Lorenz, P.E., AISC director of education and training. Accordingly, the 1995 seminar series will be divided into four areas: *The New 50 ksi Steel; LRFD for the Practicing Engineer*; *Learning from Northridge*; and *Answers to the Most Commonly Asked Questions*.

Work is currently underway for the development of a new 50 ksi yield strength steel specification that will replace ASTM A36 as the industry base standard. This new steel will be designed to improve performance with better defined strength and material limits. Part One of the AISC 1995 Steel Seminar is designed to answer engineers' and fabricators' questions about this new material's effect on design and construction. Included will be a discussion of minimum and maximum material strength, ductility and economics. "The shift to the 50 ksi base material as the preferred material is intended to simplify and improve design practice," according to Lorenz.

Part Two of the 1995 Steel Seminar will focus on simple, straightforward procedures for designing members and connections with the 1994, 2nd Edition LRFD Manual of Steel Construction. A recent Gallup survey commissioned by AISC revealed that most engineers acknowledge that LRFD is the Specification of the future and that it is only a matter of time before most engineers switch from ASD to LRFD. Additionally, AISC has recently issued a position statement unequivocally reaffirming that LRFD is the preferred Specification for the fabricated structural steel industry. As an added bonus, attendees at the seminar will receive a copy of an LRFD design-aid software program.

Next up is a discussion of the lessons learned from steel performance during the Northridge Earthquake. Preliminary studies indicate that alternatives are available to avoid moment frame damage during a seismic event. This portion of the seminar will focus on these alternatives, as well as the latest research and code changes. In addition, a discussion of overstrength/redundancy in steel design will be presented. "Newly created research is aimed at sorting out the complexities of actual seismic performance," Lorenz stressed.

Finally, the seminar concludes with a 45-minute presentation of answers to the most commonly asked questions received by AISC's engineering staff. AISC's staff engineers, in addition to their work on manuals, Specifications and other design aids, routinely field calls from practicing engineers, fabricators and erectors. The most common of these questions—dealing with such topics as bolt installation, painting, and tolerances—have been compiled. The seminar series is currently scheduled to reach 34 cities, beginning with Charlotte on June 20 and concluding with Orlando on November 30. Each seminar begins at 2:00 p.m. and ends at 8:15 p.m. Cost for the seminar, which has a CEU value of 0.45, is $120 ($90 for AISC members). The fee includes the lectures, numerous handouts, LRFD educational software, and dinner.

For more information, call AISC at 312/670-5420 or fax 312/670-5403.

1995 STEEL SEMINAR SCHEDULE

June 20...............Charlotte
June 22...............Greenville
June 27...............Raleigh
June 29...............Richmond

July 25................Minneapolis
July 27................St. Louis

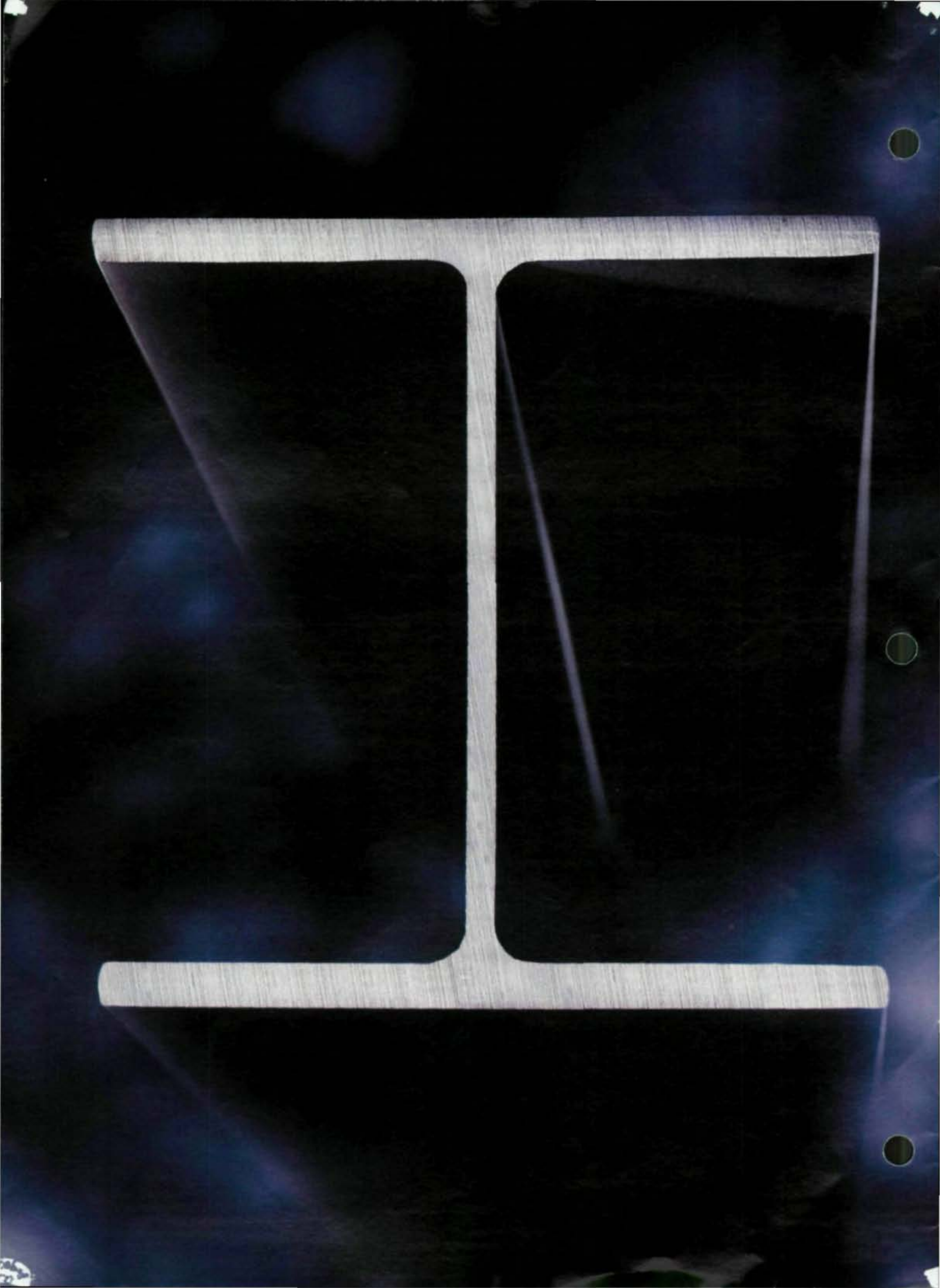
August 8.............Rochester, NY
August 10...........Albany
August 15...........Washington, DC
August 17...........Philadelphia
August 22...........Chicago
August 24...........Milwaukee

September 7........New York City
September 12.....Meriden
September 14.....Boston
September 19.....Dallas
September 21.....Houston
September 26.....Denver
September 28.....Kansas City

October 3...........Birmingham, AL
October 5...........Atlanta
October 10.........Detroit
October 12.........Indianapolis
October 17.........Cleveland
October 18.........Columbus
October 19.........Cincinnati
October 24.........Memphis
October 26.........Nashville
October 31.........Pittsburgh

November 2.......Edison
November 7.......New Orleans
November 9.......Albuquerque
November 28.....Miami
November 30.....Orlando

CALL 312/670-5420 FOR INFORMATION OR A REGISTRATION FORM



THIS IS GOING TO BE THE MOST POPULAR CONSTRUCTION SIGHT IN AMERICA.

Nucor-Yamato Steel is reshaping the way America sees construction. With an expanded mill that can supply more structural steel than any mill in the Western Hemisphere.

THE BIGGEST MILL And not only more steel, but a broader product line with beams up to 40 inches deep. Perhaps best of all, we are a low cost producer. It all adds up to just one thing: talk to Nucor-Yamato before you decide to build your next project.

AVAILABILITY With 1.6 million tons behind us, nothing can stop our availability and on-time delivery by rail or truck or barge on a wide range of products. In fact, we can offer high strength steel at little or no more cost than standard ASTM A36 beams.

QUALITY Nucor-Yamato is a state-of-the-art mill, including electric arc and ladle metallurgy furnaces, continuous beam blank casting and high performance rolling mills designed to produce stock and other lengths for all our customers. All our steel is low carbon and fine grain with superior toughness and surface quality that offers both weldability and improved corrosion resistance.

LOW COST PRODUCER Perhaps best of all, it's produced cost effectively for some of the most competitive prices you'll ever see. And if you're in the construction business these days, that's a sight for sore eyes.

Contact Nucor-Yamato Steel early in your next project at 800/289-6977, FAX 501/763-9107; or write to us at PO Box 1228, Blytheville, Arkansas, 72316.

Structural shapes from W6 X 15 to W40 X 397
Standard sections including C15 X 33.9-50, MC 12 X 31-50, MC 13 X 31.8-50 and MC 18 X 42.7-58
H-Piling ranging from HP 8 X 36 to HP 14 X 117
Grades include ABS Grades A, B, AH32, AH36; ASTM A36; ASTM A572 Grade 50; ASTM A588 ("weathering steel"); ASTM A709 (AASHTO M270 equivalents); as well as CSA 40.21 Grade 44W

NUCOR-YAMATO STEEL COMPANY

Please circle # 39

DESIGNING PETROCHEMICAL FACILITIES IN STEEL

Ease and speed of construction are important factors in the emergence of steel for petrochemical plants

A typical pipeway

By Mehmet Tan, S.E.

DUE TO SHORT SCHEDULES THAT INCREASINGLY INCLUDE CONTRACTUAL INCENTIVES TO FINISH WORK AHEAD OF SCHEDULE, steel—with its faster speed of erection—is rapidly becoming the material of choice for petrochemical facility design. Depending on the construction schedule, labor costs, and site locations, petrochemical plant design typically utilizes one of two types of steel construction: conventional (or stick building) or modular.

CONVENTIONAL CONSTRUCTION

Using conventional construction techniques, steel structures are erected and installed from the base to the top, supported on concrete foundations. Because of its versatility and design simplicity, conventional stick construction is the most common type of construction for petrochemical projects.

A typical refinery or petrochemical plant is comprised of one or more units. Each unit consists of several structures. In a plant, the steel structures commonly range from simple single-column pipe supports to more complex heavy structures supporting large reactors and vertical/horizontal vessels. The structures supporting reactors may be as tall as 200 ft.

Engineering of structures in refineries often requires consideration of non-traditional loading or operating conditions. In pipe support structures, for example, additional loads associated with the anchorage of pipes against thermal growth and friction forces have to be studied. Vibration limitations of structural systems supporting vibratory equipment (e.g. pumps, compressors, etc.) may also be a design consideration. To minimize overstress in the piping that runs from the structures to various equipment, a detailed study of the expected actual deflection of the structures due to the wind and seismic forces may be required. For seismic design purposes, the majority of the petrochemical and refinery structures are classified as non-building structures. The Uniform Building Code (UBC) provides special provisions for the design of these structures.

For comparison purposes, the cost of a typical refinery or petrochemical unit is in the range of one to two hundred million dollars. The amount of steel in such a facility can be on the order of 1,500 to 2,500 tons.

MODULAR CONSTRUCTION

Although the term "modular construction" has long been familiar in offshore construction (i.e. oil and gas production and process plants constructed in shallow and deep water ocean locations), this technique has become increasingly attractive for many projects involving construction of onshore, land-based facilities.

Modular construction is used when requested by the client in an effort to reduce field, manpower or construction time. The feasibility of using modular construction also can be evaluated by the engineer/contractor when looking at the potential benefits of modularization:

- cost savings
- schedule reduction
- reduced social and environmental impact
- improved labor relations
- reduced project risk
- improved quality control

Additionally, it may not always be economical or feasible to use conventional construction techniques. For example, the construction window for plants built on the Alaska North Slope is approximately three months. In such cases, it is advantageous to have most of the structures designed, fabricated and preassembled as modules prior to jobsite delivery.

In addition to reducing field construction time, modular construction will reduce the field manpower requirements. This is a benefit for projects facing either high labor costs or the unavailability of skilled craft labor near the construction site. Another means of reducing field labor is by utilizing a subassembly technique, whereby equipment, structure and piping can be prefabricated into larger blocks for final assembly at the jobsite. Modularization can be either full or partial, and it is important to recognize how much modularization will best benefit a particular project.

On a modular project, the first step is to determine the methods of transportation (land and sea), handling and placement of the modules. The location of the fabrication yard and the shipping routes can then be established. Typically, it is most economical if the fabricator is close to water to allow for shop assembly and immediate loading of sections onto barges. Inland fabricators, however, can still be utilized for modular construction. In that case, members are fabricated and shipped to a modularization yard where the steel members are assembled. It is common to have several fabricators working in conjunction in a modularized construction project to further speed deliveries.

The next step is to identify design loads, taking into account environmental conditions that affect the modules during shipment, handling and transportation to their final location.

Because of the special loading conditions imposed on the modules, modularized structures generally require more steel than on-site stick construction. The additional steel usually is in the form of additional bracing members or stiffened/larger members that help maintain the integrity of the structure during transportation and installation. As a result, a heavier base frame may also be required.

Modularized assemblies are often shipped by sea

Structural Framing Systems

The structures in refinery and petrochemical projects usually are designed as moment frames, braced frames or a combination of the two. Moment frames are mainly used in areas where access to the underside of the structure is required. Examples are structures with equipment underneath that requires periodic maintenance inspection/removal and structures built over access roads. In areas where access is not a problem, braced frames are commonly used.

Most refinery structures are open frame types (no siding). In platforms, the walking surfaces are either checkered plates or grating. Safety handrails are provided around the platform areas. Stairways or ladders, with or without cages, are used to access various platforms.

Connections

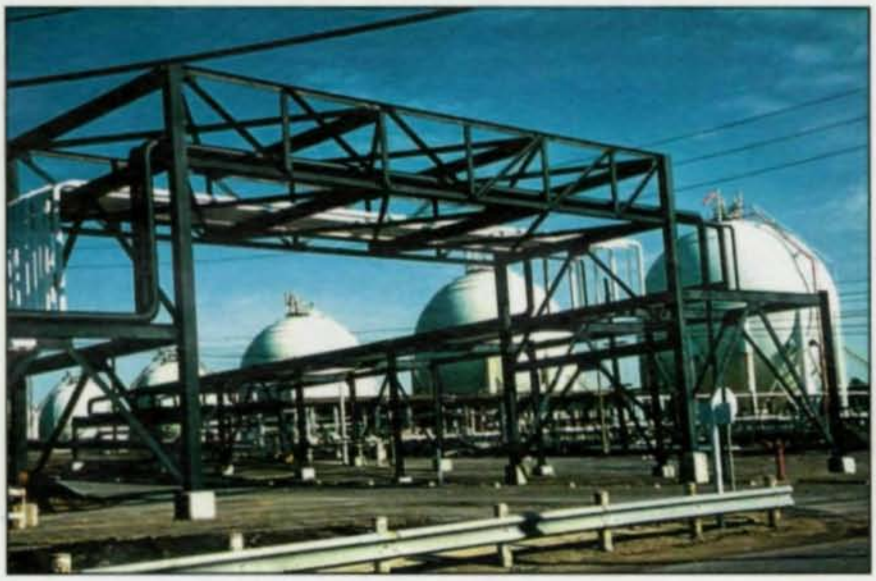
Beam-column moment connections are either bolted end plate type or full penetration welded. Shear tabs are commonly used for connection of the floor beams. For the heavy load-carrying members and members subjected to significant axial loads, double-angle standard AISC connections are used. High-strength bolts (bearing type) with washers are used throughout the structures (snug tightened). At the bolted beam-column moment connections and at the joints with bolts subjected to tension forces, bolts are fully tensioned.

Corrosion Control

Due to severe corrosive conditions in refineries and petrochemical facilities, all steel members in such plants—including connection bolts, checkered plates, grating, ladders, stairways and handrails—are either galvanized or painted. Until recently, it has been common to use painted steel in most refineries. The latest pricing comparisons, however, indicate that galvanizing is the relatively less expensive alternative, especially when considering the higher maintenance cost for painted steel compared with galvanized steel. Other factors that are taken into account when determining whether to galvanize or paint include the galvanizing capabilities of the fabricators in regards to the size of the steel members and the proximity of the galvanizer to the fabrication yard.

Fireproofing

In process areas of the refineries and petrochemical plants, most of the piping and equip-



Typical equipment access bridge

ment contains hydrocarbons, which are highly flammable. To limit damage due to fires in such areas, the structures supporting these pipes and equipment generally are fireproofed. For this purpose, concrete is commonly used as a fireproofing material. There also are some pre-mixed cementitious light-weight fireproofing materials, such as Fendolite and Pyrocrete, which may be used based on economic considerations. On the one hand, concrete permits attachment of small bore pipes and electrical cables to the fireproofing. On the other hand, Fendolite and Pyrocrete are lighter than concrete and their usage will result in the size and weight reductions for the steel members (an especially important consideration in high seismic zones).

In typical petrochemical applications, the fireproofing is not covered with wallboard or other architectural coverings, so the fireproofing must be of an exterior grade and consistent appearance becomes important.

Steel Pricing

Because much of the work in petrochemical construction is executed in a design/build mode, the entire scope of the work is not known in the bidding process. When going out for bid, fabricators are typically provided with standard steel details, examples of typical structures and an estimated steel tonnage. However, the actual drawings are not available.

At the beginning of the project, when pricing is obtained, the fabricator is usually asked to bid a unit price for steel based on the items that have been supplied. The request for unit pricing is broken down into very specific detail, in several different categories of structural steel. These categories are then typically broken down into specific member weights. For example:

General — Structural steel with most detailing required for ladders, stairs, platforming, handrails, etc. Most equipment support structures are in this category. Also included are miscellaneous components and labor charges.

- light steel (0-20 lb./ft.), painted/galvanized
- medium steel (20.1-40 lb./ft.), painted/galvanized
- heavy steel (40.1-60 lb./ft.), painted/galvanized
- extra heavy steel (over 60 lb./ft.), painted/galvanized
- built-up sections, painted/galvanized
- ladders with cages & without

cages, painted/galvanized

- stairways, painted/galvanized
- raised pattern floor plate, painted/galvanized
- serrated grating, painted/galvanized
- handrails, painted/galvanized
- extra fasteners (A325 & A307), painted/galvanized
- rework, shop detailing (per hour) and shop fabrication (per hour)
- inland freight/load, painted/galvanized/fireproofed steel

General fireproofing — same as above, priced for light, medium, heavy and extra heavy members only.

Pipe supports/pipe bridges — relatively little detailing required, typically comprised of moment frames and/or braced frames with little or no hand rail, ladders or platforming.

- light steel (0-20 lb./ft.), painted/galvanized
- medium steel (20.1-40 lb./ft.), painted/galvanized
- heavy steel (40.1-60 lb./ft.), painted/galvanized
- extra heavy steel (over 60 lb./ft.), painted/galvanized

Pipe supports/pipe bridges, **Fireproofing** — same as above, including fireproofing

Shop assembly of equipment support platforms

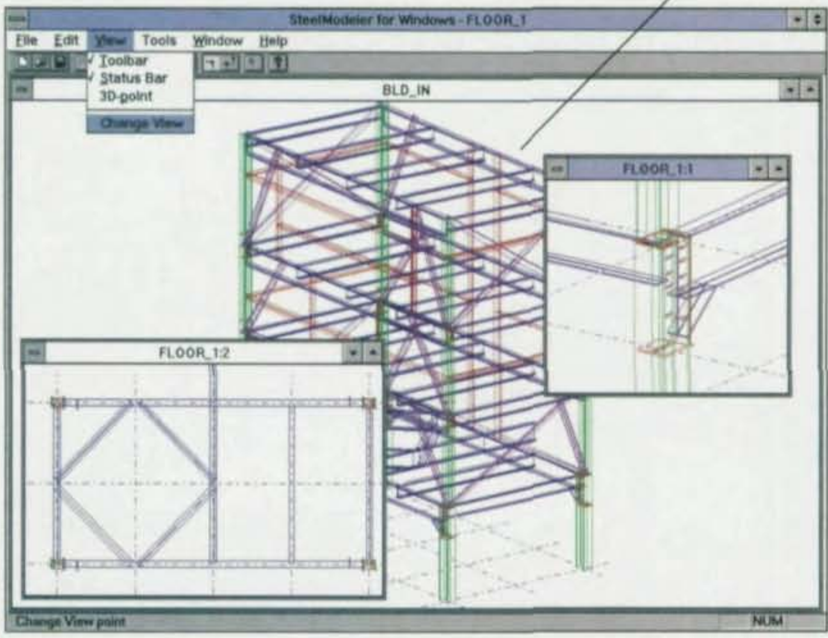
- light steel (0-20 lb./ft
- medium steel (20.1-40 lb./ft
- heavy steel (40.1-60 lb./ft
- extra heavy steel (over 60 lb./ft

It also is becoming increasingly common to solicit a price quote for submitting the structural steel drawings and details directly by electronic data interface.

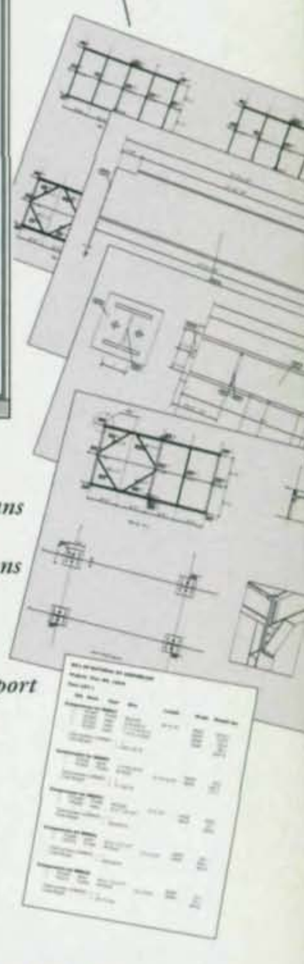
This article was condensed from a paper to be delivered at the National Steel Construction Conference. Mehmet Tan, S.E., is technical director for Fluor Daniel, Inc., in Irvine, CA.

You do the thinking . . . SteelModeler does the rest.

You create a single model.



SteelModeler automatically generates the views, shop drawings and material lists you require.



At last! A truly interactive modeling system designed for structural steel detailers.

SteelModeler is efficient, accurate, and highly productive.

SteelModeler lets you concentrate on producing a single 3D model, while it takes care of everything else.

You define your structure by selecting steel shapes from a comprehensive built–in data base and make connections using your own custom library of macros. And as you work, **SteelModeler ensures that all components fit.**

There's no need to juggle drawings of plans, sections or components. SteelModeler automatically generates any view you want on the multi–window screen, and whenever you're ready, drawings, shop details and materials lists for the whole model or any section or part you require – all in a form customized by you to your specific standards.

Revisions or modifications? Simple. You change the model, and SteelModeler automatically revises all related drawings and lists to ensure total accuracy.

- *Shop drawings*
- *Anchor bolt setting plans*
- *Erection diagrams*
- *Floor/roof framing plans*
- *Connection details*
- *Perspectives*
- *Bills of materials*
- *Neutral file import/export*

SteelModeler

The tool you would have designed for yourself.

SteelModeler runs on standard PCs and Unix workstations. Versions available: MS-DOS; OS/2; Autocad; Windows 95 (summer)
For more information, contact: dcCAD; Tel: (416) 253-4848; Fax: (416) 253-4315; Toll-free: 1-800-776-4840

Please circle # 107

See us at Booths 408/410 at the National Steel Construction Conference in San Antonio

SIMPLE FRAMING CONNECTIONS TO HSS COLUMNS

A wide variety of connections can be used with HSS columns that are not classified as thin-walled

BY DONALD R. SHERMAN, PH.D.

THE USE OF SQUARE AND RECTANGULAR HOLLOW STRUCTURAL SECTIONS (HSS) as columns in buildings is booming. In addition to their aesthetic benefits, the shapes are extremely efficient as compression members. Connecting wide flange girders and beams to tubular columns, however, has presented some difficulties. Typically, designers have adapted many of the standard simple connections *typically used with wide-flange* columns, even though little data is available regarding their use with HSS columns. Two concerns with this approach are whether there is a limit state in the HSS that could govern the connection design or if local distortion of the HSS wall could reduce the column capacity.

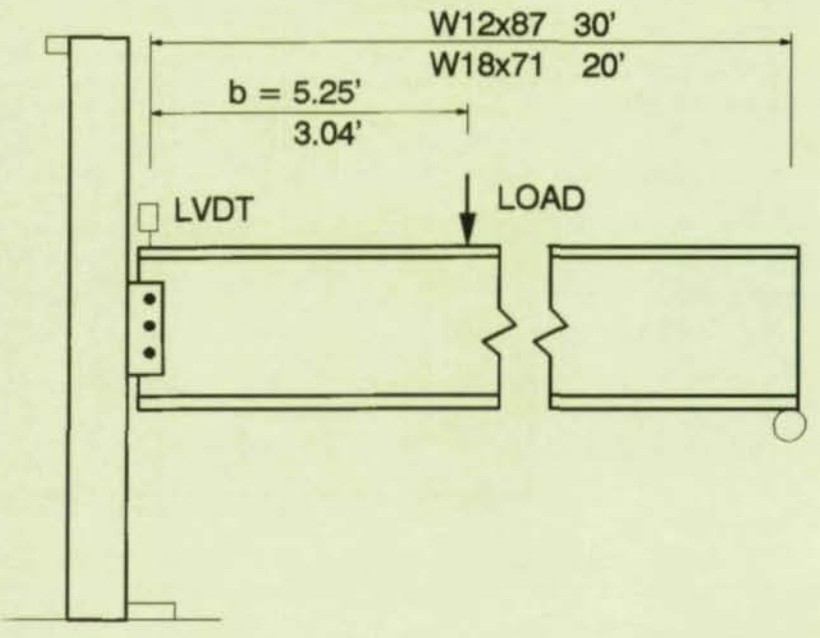


Figure 1: Basic Connection Setup

Fortunately, recent research has examined the limit states considered in the AISC Manual of Steel Construction for connections and compared them with experimental capacities. This article discusses nine different types of simple framing connections: shear tabs; through-plates; double angles; tees with vertical fillet welds; tees with flare bevel groove welds; unstiffened seated connections; single angles with L-shaped fillet welds; single angles with two vertical fillet welds; and web end plates. In all but the web end plate, the connecting elements are welded to the HSS column and bolted to the web of the wide-flange beam, with the exception of the seat angle where the flange bears on the outstanding leg. For the web end plate, the plate is welded to the beam and bolted to the HSS column using a flow-drill process that produces a tapped hole that replaces a nut in blind connections.

For each connection potential limit states are discussed and evaluated. Strain measurements indicated the relative degree of distortion in the HSS wall and data is presented to verify that the connection producing the highest strain levels in compact HSS columns does not reduce the axial load capacity. Finally, the relative economics of the various types of connections are discussed.

TEST PROGRAM

All of the connection tests used the same basic setup as shown in Figure 1. A short segment of HSS column is held vertical and the connected wide-flange beam that is simply supported at the far end is loaded with a concentrated load at a distance b from the face of the HSS. By equating the relations for shear and end rotation between a uniformly loaded beam and the test beam with concentrated load, the length of a simulated uniformly loaded beam is obtained.

$$L_u = \sqrt{2}\sqrt{L_{beam}^2 - (L_{beam} - b)^2}$$

Two different beam sizes were used to provide information on both stiff and flexible beams. The flexible W12x87 had a length/depth ratio L_u/d of 23 while the W18x71 had L_u/d of 9.8. The former was used with three bolts in the web connection, while the latter used either three or five bolts.

Tension coupons were removed from all the various types and sizes of connecting elements. (The comprehensive report includes data on yield strength, ultimate strength and actual thickness of all connecting elements and HSS columns tested.) The stress-strain curves showed a flat yield except for Tee with flair bevel welds, which had a rounded curve.

The symbols used to designate the specimens convey a great deal of information. The first digits in the symbols are used to designate the b/t of the HSS. The following letters indicate the type of connection (e.g. EP for end plate and DL for double angle). Next, a digit gives the number of bolts (three or five). Finally, a the F or S designates a Flexible or Stiff beam.

In several cases, the connecting material had a higher yield strength than expected in the planning stages. Therefore, most tests were terminated when the yield strength of the beam was reached. The relative vertical displacement between the face fo the HSS column and the end of the beam was measured with a displacement transducer. This displacement was essentiallythe shear and bearing distortion of the connecting element. For test loads reported in the tables that

Double Angle Connection Strengths (kips)

Specimen	Bolt		Connection Elements			Weld	Test
	Shear	Bear.	Yield	Rupt.	Block		
36DL3F	127	236	142	154	181	71	80+
16DL5S	212	393	242	266	293	163	147+

TEE Connection Strengths (kips)

Specimen	Bolt		Connection Elements					Weld	Test
	Shear	Bear.	Yield	Rupt.	Block	Flex. Yield	Flex. Rupt.		
36TV3F	46	70	79	78	127	187	186	126	62
36TV3S	46	66	74	73	119	174	174	126	77
16TV3S	46	66	74	73	119	174	174	126	79
16TV5S	103	157	136	136	185	547	542	215	124
36TF3F	46	81	78	91	139	185	215	126	76
16TF5S	103	181	133	157	205	537	623	269	136

are followed by a + sign, the shear-distortion curves were still essentially linear at the maximum load, indicating that a ductile failure was not imminent. For other loads, the curves had flattened in a manner typical of approaching a ductile ultimate load. It should be noted, however, that all tests showed some sign of distress in the whitewash coating on the connection.

Double Angles

For the double angle connections, no eccentricity was considered in the nominal strength of the bolts or angles and the eccentricity for the welds was the in-plane type proposed by Omer Blodgett and used in the AISC connection manuals for many years. Except for the end plate, the double angle was the strongest connecting element tested. This was due to the combined thickness of the beam web legs of the angles. Theoretically, the welds dictated the connection strength and the test loads approached or exceeded the nominal weld strengths.

Slight cracking was observed in the whitewash at the ends of the welds in both tests. For the three bolt connection, the crack was on the weld, but for the five bolt connection, it was at the toe of the weld on the HSS face. In both tests, the most extensive yielding was on the HSS legs of the angles. In the three bolt connection, the top of the angles had separated about $^3/_{16}$-in. from the HSS, while the separation was on the order of $^1/_8$-in. for the five bolt connection. No gross distortion or other indication of failure in the HSS was observed.

Tee Connections

The nominal capacity for all the Tee connections was determined by the shear strength of the bolts. These connections with welding at the edges of the Tee flanges are considered flexible in the AISC Manual of Steel Construction and eccentricity is considered in the bolts and direct shear in the welds. The test loads exceeded the nominal bolt capacities and were at the direct shear yield or shear fracture capacities of the stem. This corresponded with the extensive pattern of whitewash cracking observed on the stems. In tests 36TV3S and 16TV5S, a crack was observed at the bottom of the stem in line with the bolts as well as at the bottoms of the bolt holes, indicated the beginning of a shear rupture failure. Major cracks also were observed in the bolt holes of specimen 36TV3S.

The Tee connections were tested with the most variation in the HSS columns and beam stiffness. However, neither of these factors affected either the nominal strengths nor the condition of the HSS at the maximum test load. No distress was observed in any of the HSS faces. Separation from the HSS at the top of Tee was on the order of $^1/_{16}$-in., but there was more pushing in of the

Seat Angle Connection Strengths (kips)

Specimen	Connection Elements: Shear Yield	Connection Elements: Flex. Yield	Weld	Test
36SeF	124	64	81	50+
16SeS	124	63	81	72

Single Angle Connection Strengths (kips)

Specimen	Bolt: Shear	Bolt: Bear.	Connection Elements: Yield	Connection Elements: Rupt.	Connection Elements: Block	Weld	Test
36SL3F	80	127	82	83	92	74	58
16SL5S	133	212	140	143	152	126	134
36SV3F	80	127	82	83	92	51	75
16SV5S	133	212	140	143	152	103	133

End Plate Angle Connection Strengths (kips)

Specimen	Bolt: Shear	Bolt: Bear.	Connection Elements: Yield	Connection Elements: Rupt.	Connection Elements: Block	Weld	Test
36EP3F	127	244	159	159	192	119	79+

Shear Tab Connection Strengths (kips)

Specimen	Bolt: Shear	Bolt: Bear.	Connection Elements: Yield	Connection Elements: Rupt.	Connection Elements: Block	Weld	Test: Load	Test: Mode
5TB3F	69	111	84	91	99	134	75+	
5TB3F	69	99	91	81	92	134	74+	
5TB5S	125	178	151	134	146	223	143	A
10TB3F	69	111	84	91	99	134	73+	
16TB3F	69	111	84	91	99	134	70	A
16TB3S	69	99	91	81	92	134	70	A
16TB3S	69	111	84	91	99	134	84	D
40TB3F	69	99	91	81	92	134	71	A,C
45TB5S	125	178	151	134	146	223	132	A
45TB3F	69	99	91	81	92	134	70	A,D
45TB3F	69	81	64	66	73	134	60	A,B

MODES:
A — Significant bearing distoration
B — Shear fracture crack started
C — Punching shear in HSS wall
D — Weld crack or surface tear of HSS beneath the weld

HSS wall at the bottoms of the Tee. Flare bevel welds did not show any difference in behavior from the Tee connection with vertical welds on the HSS frame. However, slight whitewash flaking at the ends of welds were observed in 36TV3S and 36TF3F with the thinner HSS.

Seat Angles

The nominal weld resistance for the seat angle connections considered the eccentricity based on a $^1/_2$-in. setback that was actually used in the tests. Following the procedure established by Garrett and Brockenbrough as recommended by AISC, the bearing length (N) was less that 2.5 times the distance from the flange face of the beam to the toe of the web fillet (k), so that the *reaction was assumed to act at* (N + 2.5k)/4 from the end of the beam. Since the procedure resulted in a negative value of N, N was taken as zero in the flexural yield calculation.

Both tests were terminated when yielding in the beam web was observed. At this load, some yielding was observed in the vertical leg of the seat, but there was no evidence of distress in the HSS or the welds. The outstanding leg was bent so that the observed bearing length to the beam was less than $1^1/_4$-in., which was the position of the erection bolts through the flange.

Single Angles

The nominal resistance for the single angles ignore any eccentricity for the bolts. For the L-shaped AISC weld, the in-plane eccentricity was included. Since the connections with vertical welds are not an AISC standard, the weld capacities were based on an out-of-plane eccentricity to the bolt line and half the value from the AISC ultimate strength eccentric load table is reported. Using half the value conservatively assumes that only the weld at the heel of the angle is effective, since it is in line with the outstanding leg.

The two connections with the AISC-shaped weld had extensive yielding in both the outstanding and the HSS legs of the angle and small cracks were found at the bottom of the lower two bolt holes. The three bolt connection had slight whitewash cracking at the ends of the welds, but the five bolt connection indicated yielding along most of the length of the vertical weld. The vertical distortion of the connections was $^{3}/_{8}$-in. and the separation of the top of the angle from the HSS was evident at the very early stages of loading. Flexural yielding of the HSS leg is a limit state that should be considered and probably controlled the capacity.

The connections with vertical welds also had extensive yielding in both legs and the five-bolt connection had an initiation of shear rupture at the bottom of the bolt line. Both specimens had small cracks at the bottom of the bolt holes. The five-bolt connection had whitewash cracking at the top of the center weld, while the three-bolt connection had slight whitewash cracking in the HSS wall at the ends of both welds. Vertical distortions were again on the order of $^{3}/_{8}$-in. but separation was restricted by the weld.

END PLATE

The end plate connection had the highest nominal strength. The bolt and connecting element strengths were similar to the double angle but the weld strength was higher due to the lack of eccentricity.

The test was terminated due to the beam capacity, although shear yield lines were extensive, especially in the lines of the bolts.

Shear Tabs

The nominal bolt resistances are based on the eccentricities for the rigid (rather than the flexible) support in the AISC Manual of Steel Construction, since these gave results closer to the test values. Tests 5TB3F and 16TB3F are reported twice, since the specimens for snug and fully tensioned bolts had different materials for the tabs.

In addition to the potential failure modes observed in the table, all the specimens had considerable gross yielding of the shear tab. The punching shear failure of the HSS wall for 40TB3F was actually in two specimens, one with snug bolts and an identical test with fully tensioned bolts.

Strength Limit States

With the wide range of connection types variables reported, only one limit state was identified in the tube of the HSS—a punching failure when a thick shear tab was used with a thin HSS. A simple criteria to prevent this type of failure can be derived from an inequality that the yield force in a unit depth of shear tab does not exceed the through-thickness shear rupture strength on two planes of a unit length of the HSS wall.

Yield line distortion of the

$$F_{y(tab)} t_{tab} < 2(0.6\, F_{u(HSS)} t_{HSS})$$

or

$$t_{tab} < 1.2 \frac{F_{u(HSS)}}{F_{y(tab)}} t_{HSS}$$

face of the HSS was never a limit state. For a simply supported beam, there are limited end slopes that prevent unrestrained distortion of the HSS. Upon careful examination or measurement, pushing in of the HSS wall at the bottom and pulling out at the top of the connections could be observed. However, gross distortions of all the typical simple connections tested were much more evident. The limit states that controlled were those associated with connecting elements, welds or bolts.

Strain in HSS Wall

In order to determine the effect of the connection types on local distortion of the HSS column, strain gauges were mounted at the center of the wall one inch below the connecting element.

Connections such as tabs and single angles that have load transfer through a weld at the center of the HSS have the highest transverse strains. These will typically exceed yield. An exception to this is the through-plate that inherently reinforces the center of the wall. Connections with welds near the sides of the HSS have significantly less transverse strain at the center of the wall. The end plate and seat angle connections produce little transverse strain. Longer connections with five bolts produce less transverse strain that three bolt connections and HSS with thinner walls or higher b/t tend to have larger strains.

Effect Of HSS Distortion

In order to address the question of whether local distortion of the HSS has a detrimental effect on the column capacity, a series of tests were conducted to compare the influence of shear tab and through-plate connections. These types of connections represent the extremes of inducing transverse strain into the HSS wall. The conclusion of these tests is that shear tab connections used with HSS columns that are not thin-walled will develop essentially the same column strength as those where the wall is reinforced with a through-plate. With thin-walled HSS, shear tabs may have a detrimental effect on the axial column capacity. For connections on only one side of the HSS column, there is no strength reduction for using shear tabs. It is safe to assume that this conclusion holds for other types of simple connections that have smaller transverse strains.

Connection Costs

Since a number of different connection types were studied at the same time, excellent comparative cost data was available. For comparison sake, the costs were based on the least expensive connection being assigned a relative value of 1.00. The costs include connecting material and the labor to fabricate the connection, including welding to the HSS or to the beam web in the case of the end plate. Note however, that the cost of the end plate is somewhat uncertain since flow-drilling the holes is not a routine shop operation at this time. The costs also do not reflect shop preparation of the beam or field erection.

Relative Connection Costs

Single Angle, AISC Weld	1.00
Shear Tab	1.05
Single Angle, Vert. Welds	1.17
Seat Angle	1.36
Double Angles	1.50
Tee, Vert. Welds	1.62
End Plate	2.15
Through-Plate	2.25
Tee, Flare Welds	2.42

Conclusions

The tests show that the variety of simple framing connections typically used in steel construction can confidently be used with HSS columns that are not classified as thin-walled. The tabulated connections capacities and criteria for evaluating connections that appear in the AISC Manual of Steel Construction can be applied when HSS columns are used. The only additional limit state that must be considered is a simple thickness criteria for punching shear of the HSS wall when shear connections are used.

Connections that involve welding at the center of an unreinforced HSS wall will produce local strains that exceed yield. However, the resulting wall distortions are barely noticeable and not nearly as great as the distortions of the connecting elements. The local distortion in the HSS wall has negligible influence on the column capacity as long as the HSS is not classified as thin-walled. This applies to connections on one side of the HSS or symmetric on both sides.

Finally, careful consideration should be given to the type of connection specified since the connection cost can vary by a factor of 2.5.

Donald R. Sherman, Ph.D., is a professor at the University of Wisconsin. This article is based on a paper delivered at the 1995 National Steel Construction Conference. The connection and column tests were supported by the Steel Tube Institute of North America with additional funds provided by the Society of Iron & Steel Fabricators of Wisconsin and AISC. The HSS material was provided by the Welded Tube Co. of America. Special thanks is due to Dave Mathews of Ace Iron & Steel Co., who fabricated the connection material and provided cost estimates for fabrication.

Transverse Strain In HSS At 50 kips Shear (μin/in)

Type	Bolts	HSS t(in.)	HSS b/t	Strain
Tab	3	5/16	19	-3900*
Tab	3	3/16	40	-3870*
Tab	3	1/4	29	-3800*
Single Angle (Vert. Welds)	3	1/4	36	-2100
Tab	3	1/4	45	-2100
Single Angle (AISC Weld)	3	1/4	36	-1380
Tab	5	1/4	45	-1200
Tee Vert.	3	1/4	36	-1100*
Tee Flare	3	1/4	36	-1100
Single Angle (Vert. Welds)	5	1/2	16	-1050
Double Angle	3	1/4	36	-975
Tab	5	1/2	5	-900
Tee Vert.	3	1/2	16	-750
Double Angle	5	1/2	16	-600
Single Angle (AISC Weld)	5	1/2	16	-450
Tee Vert.	5	1/2	16	-380
End Plate		1/2	36	-300
Tee Flare	5	1/2	16	20
Seat		1/4	36	40
Thru-Plate	3	5/16	19	55
Seat		1/2	16	60
Thru-Plate	3	3/16	40	475
Thru-Plate	3	1/4	29	700

Column Strengths For Tabs vs. Through-Plate Tests

b/t	m	Connection	Pult/Py Two Sides	Pult/Py One Side
15	1.39	Thru-Plate, Tight	0.53	
		Shear Tab, Tight	0.51	
		Thru-Plate, Snug	0.50	
		Shear Tab, Snug	0.49	
29	0.89	Thru-Plate	0.63	0.42
		Shear Tab	0.61	0.46
40	0.60	Thru-Plate	0.58	0.42
		Shear Tab	0.45	0.42

Our Tru-Tension® bolt was designed with one very important advantage over other twist-off bolts: a hex head. So removing this bolt or checking the installation with a torque wrench is just as easy as installing it.

Using a quiet, hand-held power tool, our Tru-Tension system always provides the correct tension. It also eliminates operator error during assembly, the bolts come pre-assembled and you can even save money thanks to the quick, easy installation and lower costs for visual inspection.

But, install any button head fastener compared to our hex head fastener, and there's no turning back. Because it's almost impossible to remove a button head fastener, you may even need a welder to burn it off. That costs you time and money. If you'd installed our hex head bolt in the first place you could remove it with a simple, four dollar wrench.

What if you have to verify the proper installation of a button head bolt? Again, it's almost impossible since you can't even get a wrench on the head to test or remove it. But you can verify installation of our hex head bolt with the turn of a wrist and a calibrated torque wrench.

Clearly, this is one bolt that won't let you screw up. Our Tru-Tension system is also fully traceable to our domestic sources, like all Nucor fasteners. They're fully tested and certified, including compliance with FHWA, DOT and AASHTO specifications for bridge construction.

So, forget about cutting bolts off with a torch if something goes wrong. Call us at 800/955-6826, FAX 219/337-5394 or write PO Box 6100, St. Joe, IN 46785 to find out more about our Tru-Tension system. And get a fastener that has a good head for business.

NUCOR FASTENER
A Division of Nucor Corporation

Please circle # 38

STEEL ERECTION AWARENESS: AN ERECTOR'S VIEW

Simple steps a design engineer can take to make an erector's life safer and simpler

New safety rules could have an effect on such areas as fall protection. Shown above is an iron worker transferring his lines.

By William G. Zimmerman II

THE MOST GENERAL ADVICE FOR ANY STEEL ERECTION PROJECT IS THAT IT'S ALWAYS BETTER TO WORK in a shop than in the field. From an erector's viewpoint, the first order of business for any design is to keep the erection as simple as possible. Doing work in the field out of position or in bad weather can lead to a need for rework or, even worse, to poor or unsafe construction. If it can be done in the shop, do it there. Usually the cost and safety issues are greatly diminished.

WELDING ISSUES

The quality and dependability of field welding has improved greatly over recent years. Still, some types of welding, specifically full penetration, should be avoided where possible. Full penetration field welds require a highly skilled and qualified welder and have to be inspected, usually with ultrasonic testing. Both of these factors increase the cost of a project and extend the erection schedule. Therefore, an economic design will use full penetration welds only where no other means is practical.

The size of the weld also is important, because multipass (where the welding process is repeated on top of a previous weld) welds are more time intensive and, again, require more highly trained and skilled welders. Also, many multipass welds require additional inspection beyond just visual. As an example, in the case of fillet welds, any size larger than $^1/_4$ to $^5/_{16}$ in. will require multipass welds. The requirement for continuous welds where stitch welds will suffice increases cost and many times causes distortion problems. If continuous welds are used for a seal, consider caulking with a high quality sealant instad.

Weld configuration is another area that should be considered. A fillet weld is the easiest to produce and does not require any special preparation. This weld can be produced by any certified welder. Inspection is simple and can be visual for the single pass configuration.

Finally, the position of the weld is another area where savings and quality can be improved. Using a flat position is always easiest for the welder.

CONNECTION DESIGN

Erectors always prefer a connection that can be made in the setting of the members that is open to access and positive, such as shear tabs and seats. When members have to be knifed through the adjoining members connection, safety and cost become a serious issue.

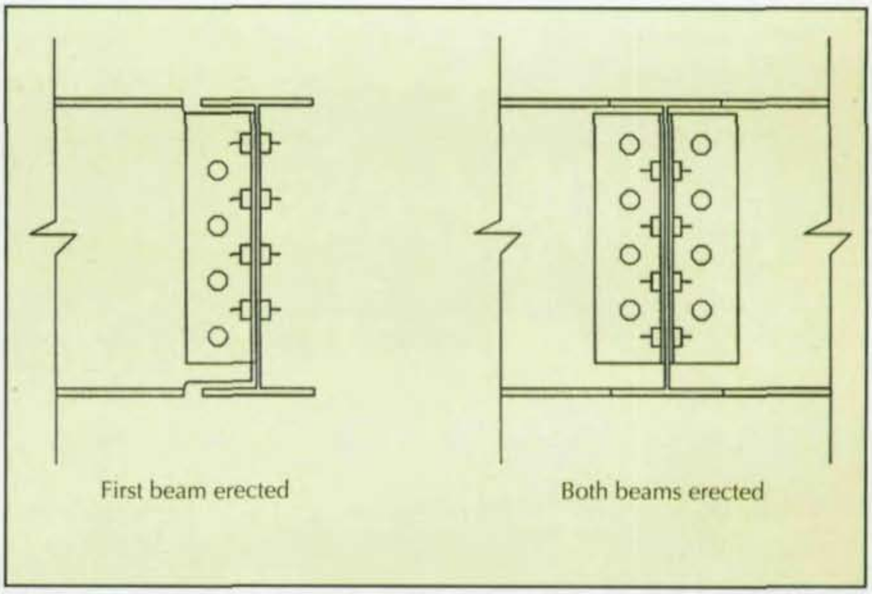
An easy way to reduce field labor is to focus the connection work on similar members. The erector is able to set up standards and develop techniques to speed up the erection process.

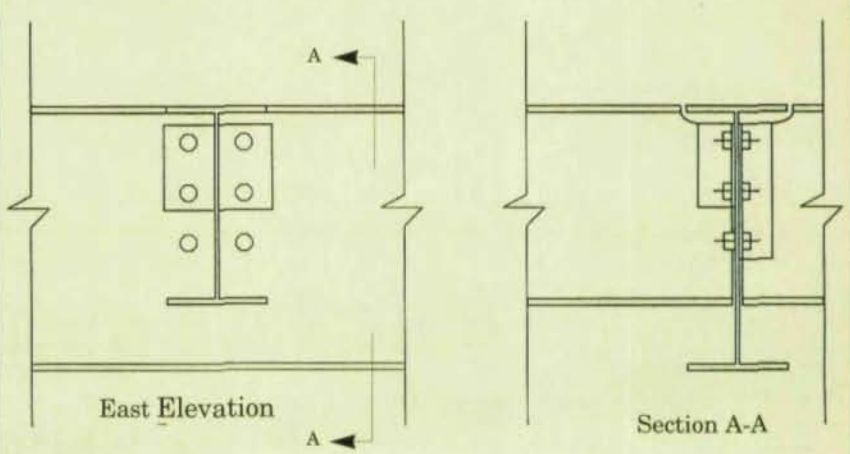
Another productivity tool is the use of twist-off bolts. One worker using a 110-volt shear wrench can tighten bolts much faster and with better quality than before. When the spline is no longer on the bolt, that bolt has been torqued to the proper tension. There is, however, an offset when using tension set bolts. The pattern used in tightening the connection must be followed or the top bolt will be loose when the bottom bolt is tightened. Utilizing the proper sequence in tightening the bolts will negate this problem. Note, however, that some erectors may prefer the turn-of-nut method, the use of calibrated wrenches or the use of direct tension indicators. For the most economical design, check with the erector on a specific project.

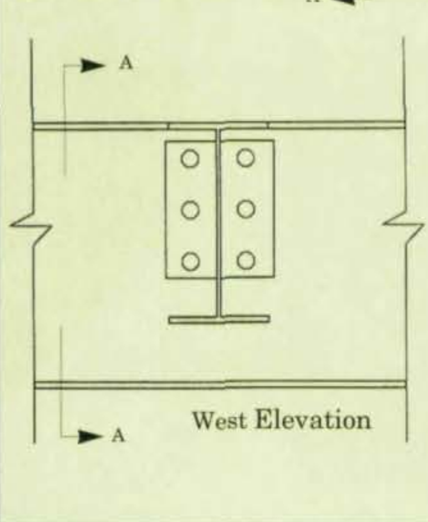
OSHA, through the Steel Erection Negotiated Rulemaking Advisory Committee (SENRAC) is expected to issue new recommendations on Subpart R (29 CFR 1926.750) on August 6. With the Subpart R ruling, OSHA will probably not allow any connections greater than two thicknesses in the setting process. Members that require multi-thicknesses will need to have seats or a two-ply bolt connection for setting. Multi-ply connection will be allowed after the initial setting is complete.

Beam And Column Design

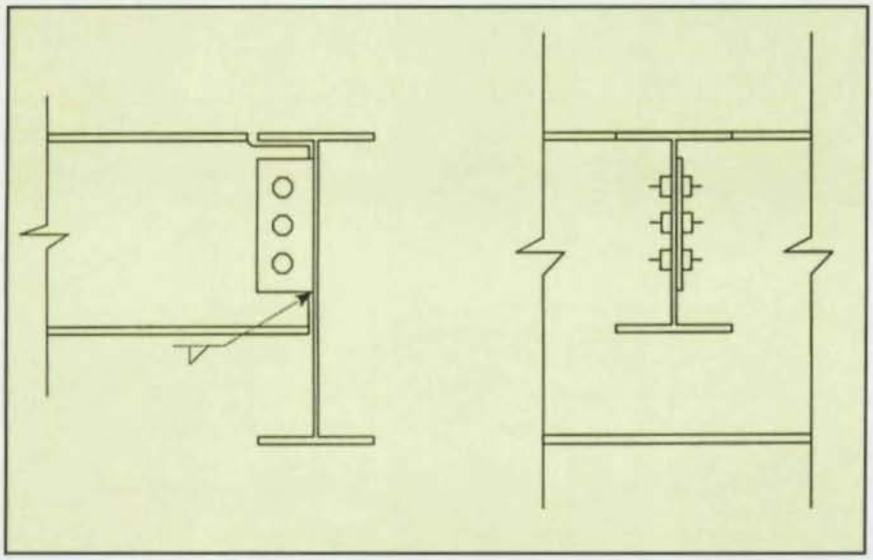
Light, long beams are dangerous to install, especially if a number of holes are punched in the flanges. This type of beam tends to roll on itself and buckle when hoisted or when set in place without any lateral stabi-







Shown at top is an example of a double (ply) connection that may be outlawed by the new OSHA safety requirements. The connection at left and above has one double ply, which is acceptable to erectors, but would require a certain erection sequence.



Shear tabs, such as the one shown in the detail above, are the preferred connection in Colorado.

lization. An example size is W12x16 with flange width of 4-in. and thickness of $^1/_4$-in. over 20-ft. in length.

Long light beams used in a cantilevered configuration also are a safety hazard. The erector must, at some time, climb out on that member before it is properly braced, which can lead to an accident. While one goal of the design engineer is often to use the lightest members possible, it is important to remember that fabrication and erection costs are not always weight dependent. In using more weight in the proper places, savings in erection are possible that will more than compensate for the added material costs of the member.

Columns longer than two stories usually require special handling and the savings from not needing a splice is usually lost by the need for special field handling.

BAR JOISTS

A quote from a letter from AISC-Associate Member Vulcraft, dated Nov. 1, 1994, states: "The Steel Joist Institute has published the 40th Edition of the Standard Specifications, Load Tables and Weight Tables for Steel Joists and Joist Girders. These new specifications have extensively changed the handling and erection stability requirements for K-Series and LH-Series Joists. These new requirements significantly change bridging design and will become effective industry wide on November 1, 1994." This action is the result of safety in erector requirements brought up under SENRAC.

SLIPPERY SURFACES

Research has shown that many accidents in the field occur on slippery surfaces, such as decking, beams and columns. SENRAC is focusing on the paint or galvanized treatment given to these surfaces in the plant. Coefficients of friction that would be minimums for the product are under discussion and product surfaces may be changed to meet the set requirements.

The construction process demands a team effort to be totally successful. The erector is only one member of that team, but, by utilizing the knowledge of your local erector in the design process, you can save money, tighten schedules and make the erection of a steel-framed building a safer process.

William G. Zimmerman II is president of AISC-member Zimkor Industries, Inc. in Littleton, CO. This article is based on his many year's experience, primarily with buildings up to six stories, and is part of a paper to be presented at the 1995 National Steel Construction Conference in San Antonio.

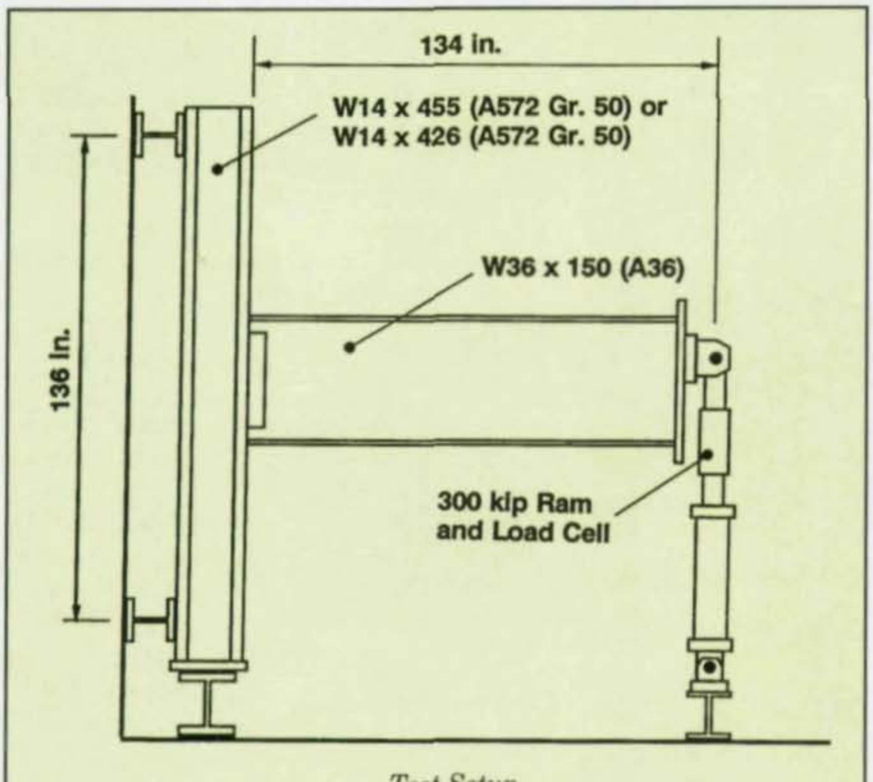
WITH VULCRAFT ON DECK, THIS

The Ballpark at Arlington, home of the Texas Rangers, built with Vulcraft composite deck.

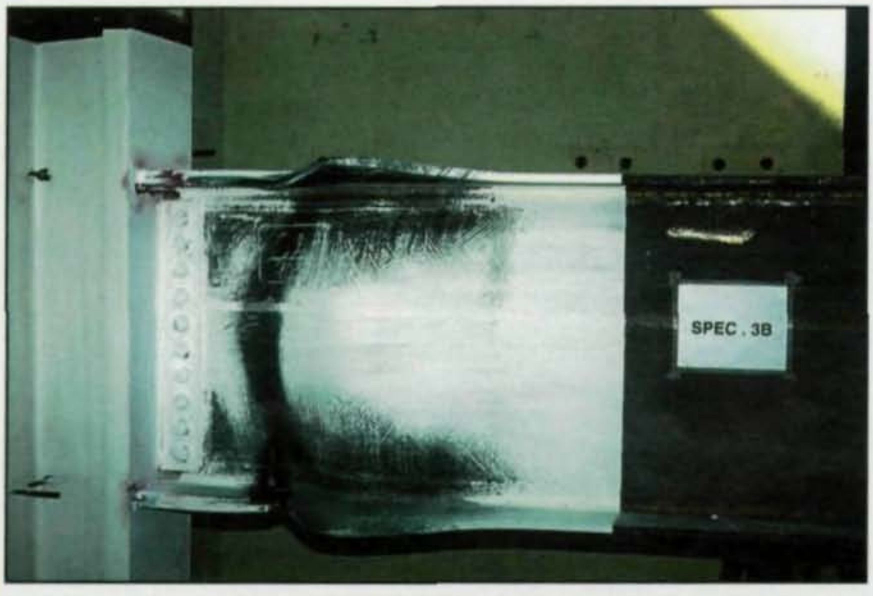
TESTING CONNECTIONS

An overview of the AISC Northridge Moment Connection Test Program

By Michael D. Engelhardt, Thomas A. Sabol, Riyad S. Aboutaha, and Karl H. Frank



Test Setup



Specimen 3B after testing.

THE JANUARY 17, 1994 NORTHRIDGE EARTHQUAKE CAUSED SIGNIFICANT AND LARGELY UNEXPECTED DAMAGE to steel moment frame joints. Within three months, a short term intensive testing program was initiated.

Damage to steel moment frames in the Northridge Earthquake was associated primarily with the welded flange-bolted web moment connection—a design widely used throughout the West Coast. The expected behavior of this detail under severe earthquake loading is a ductile response associated with flexural yielding of the beams or shear yielding of the column panel zones, without the occurrence of fracture at the beam-to-column connections. In effect, the connection should be stronger than the beam or panel zone, thereby permitting the beam or panel zone to develop the fully yielded and strain hardened strength associated with the development of large ductility. The connection must exhibit the actual strength of the adjoining frame elements, considering such factors as yield stress values in excess of minimum specified values, strain hardening, and the effects of a composite floor slab.

Contrary to the expected behavior, a variety of different types of fractures were observed in the moment frame joints in a large number of steel buildings examined after the Northridge Earthquake. Fractures in the immediate vicinity of the beam flange groove welds were a com-

Test Connection Details

Spec.	Connection Description	Beam Top Flange Reinforcement	Beam Bottom Flange Reinforcement	Web Connection	Continuity Plates
1	Standard Welded Flange-bolted web	None	None	8- 1" A325 bolts + supplemental web welds	None
2	All-welded	None	None	Beam web directly welded to column flange with cjp groove weld (4-1" A325 erection bolts provided	None
3	Reinforced with cover plates	.75" thk. x 12" wide tapered cover plate	.625" thk. x 14" wide rectangular cover plate	10-1" A490 bolts + supplemental web welds	None
5	Reinforced with cover plates	1" thk. x 12" wide tapered cover plate	1" thk. x 12" wide tapered cover plate	10-1" A490 bolts + supplemental web welds	1" thk. full depth
6	Reinforced with ribs	two-1.5" thk. x 5.5" high tapered ribs	two 1.5" thk. x 5.5" high tapered ribs	10-1" A325 bolts + supplemental web welds	1" thk. full depth
7,8	Reinforced with cover plates	.75" thk. x 12" wide tapered cover plate	.625" thk. x 14" wide rectangular cover plate	10-1" A490 bolts + supplemental web welds	1"thk. full depth
9	Side-strap	not applicable	not applicable	9-1" A490 bolts + supplemental web welds	2" thk. full depth

mon form of observed damage, with fractures near the interface of the beam flange groove weld and column flange being particularly common. Fractures of the column within the joint region also were observed. These included the pull out of "divots" from the column flange at the groove weld, as well as fractures running across portions of the column flange and web. In some instances, fractures passed through the full depth of the column section. Fractures occurring at or initiating from the beam bottom flange groove weld appear to have occurred far more frequently than at the beam top flange.

None of the connection damage resulted in the collapse of a steel moment frame building, nor did it result in any loss of life. However, this damage is contrary to the design intent and expectations of an earthquake resistant steel moment frame. The purpose of the AISC test program was to evaluate potential improvements to the connection design.

Causes of the observed damage have been the subject of considerable discussion. While it will likely be some time before all the contributing factors are clearly identified and understood, it was necessary, for the purposes of the AISC test program, to speculate on the causes of the connection damage. Some of the factors considered for the purposes of the AISC test program include:

Selected Data For Flat Position "Field" Groove Welds

Spec.	Electrode Diameter	Electrode Designation	Top Flange Back Up Bar	Bottom Flange Back Up Bar
1	.120"	E70T-4	Removed[1]	Removed[1]
2	.120"	E70T-4	Removed[1]	Removed[1]
3	.120"	E70T-4	Left in place	None used[2]
5	7/64"	E70TG-K2	Left in place	None used[2]
6	7/64"	E70TG-K2	Left in place	Removed[1]
7,8	7/64"	E70T-7	Left in place	None used[2]
9	7/64"	E70T-7	Left in place[3]	Left in place[3]

Notes: 1. After back up bar was removed, a 5/16" overhead fillet was provided at the root of the groove weld, using a0.072" diameter E71T-8 electrode.

2. Bottom cover plate groove weld served as acking for beam bottom flange groove weld.

3. Data in table is for groove welds connecting 2"x5.5" "side straps" to beam flange extensions.

Welding related factors

- inadequate welding workmanship and inspection
- lack of adherence to written welding procedure specifications
- the notch effect created by left-in-place backup bars
- use of weld metal with low notch toughness

Design related factors

- overstress of beam flange groove weld and surrounding base metal regions due to inadequate participation of the bolted web connection in transferring bending moment
- uneven distribution of stress across the width of the beam flange groove weld
- highly restrained areas within the joint developing biaxial and triaxial states of tension, thereby inhibiting ductile material response
- increase in bottom flange stress and strain due to presence of composite floor slab

Material related factors

- actual yield stress of A36 beams considerably in excess of 36 ksi
- inadequate through-thickness strength or ductility of column flanges
- inadequate notch toughness of

Example of cover plate reinforcement

column material

- high values of yield-to-tensile strength ratios (F_y/F_u)

The above list is still speculative; the roll of these and other contributing factors has yet to be definitively demonstrated.

TEST PROGRAM

The AISC testing program was directed towards steel moment frames that were under design or construction at the time of the earthquake and was intended to provide immediate data that would permit these projects to proceed at a higher level of confidence in their moment connections than would have otherwise been possible. Thus, the objective of this program was to develop interim guidelines for improved connection details in as short a time as possible.

In order to guide the program, AISC organized an advisory group representing a broad range of expertise and including researchers, structural engineers, fabricators, erectors, steel mill representatives, welding specialists, and welding inspection and NDT personnel.

Tests were conducted on single cantilever-type test specimens. Slowly applied cyclic loads were applied at the beam tip, subjecting the connection to cyclic bending moment and shear. A beam lateral support was provided at a distance of 7 ft. from the face of the column. Despite the limitations, including that the test does not include the effects of a composite floor slab or simulate the actual strain rates of a real earthquake, it is believed that this simple testing approach provided the best opportunity to collect meaningful data in a short time frame.

All test specimens were constructed with W36x150 beams and A36 steel, and either W14x455 or W14x426 columns of A572 Gr. 50 steel. (Note: while these member sizes are not atypical for moment frames in southern California, neither are they fully representative. Therefore, the possibility of differences in performance based on members sizes should be considered when viewing the results of these tests. Also, the member sizes used in this test program resulted in joints with a very strong panel zone, so inelastic action at the joint was forced into the beam.)

Example of rib reinforcement

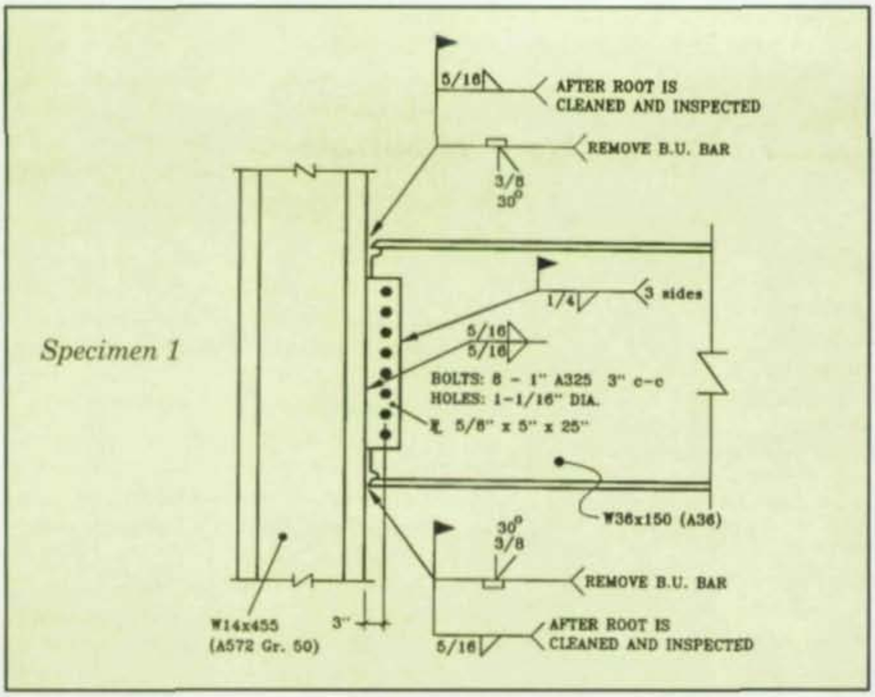
The specimens were subjected to symmetric loading cycles, with the beam tip displacement increased until failure of the connection occurred, or until the limits of the testing apparatus was reached. Because this does not replicate the actual load history of an earthquake, some judgment was required in interpreting the tests. The primary criteria used to judge test specimen performance was the plastic rotation developed by the beam. Target beam plastic rotations on the order of plus/minus 0.02 to plus/minus 0.03 radian were established. Also, total dissipated energy and the nature of the specimen's failure mode were considered in evaluating the success or failure of a particular specimen.

Test Specimens

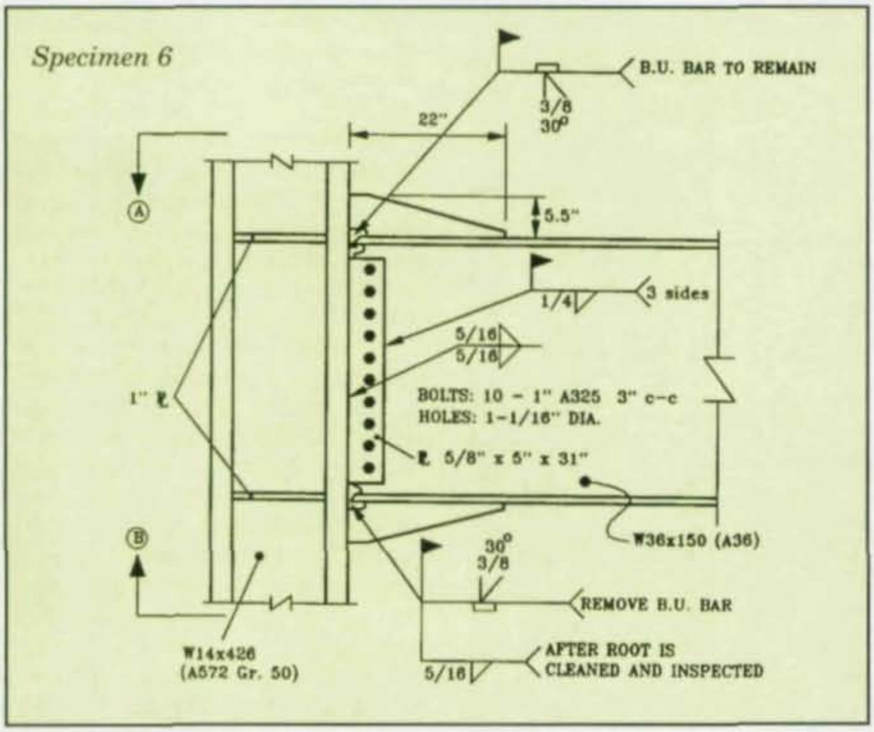
The overall approach used in the design and construction of the test specimens was to develop improvements both to the welding and to the design of the connection. Improvements were incorporated in the beam flange groove welds in an attempt to increase the level of stress that the weld region could sustain without fracture. At the same time, the connection design was modified to reduce the level of stress on the beam flange groove welds and surrounding base metal regions. As noted earlier, there are several material-related factors that may have contributed to the damage. However, no attempt was made in this test program to address those issues. Therefore, no attempt was made to set upper limits on beam Fy, specify through thickness properties or toughness requirements for the column flanges, etc. It was believed that, at least in the short term, significant changes in steel material specifications is not a practical alternative.

Nine different connection designs were considered in the test program, with two replicates (designated A & B) constructed of each by a different fabricator. (Note: ultimately, specimens 4A and 4B were not tested, so no information on these two specimens is included in this report.)

• Specimen 1: Standard welded flange-bolted web detail, designed in accordance with Section 2211.7.12



Specimen 1



Specimen 6

of the 1994 UBC. Although the conventional connection detail was used, several improvements were incorporated into the groove welds. The purpose of this specimen was to determine if the standard detail, with minor welding improvements, was likely to provide satisfactory performance.

Specimen 2: All-welded connection. Similar to Specimen 1, except the beam web, rather than being bolted, was welded directly the column. Past test programs have typically shown significantly better performance from all-welded connections. This better performance has been attributed to the improved ability of the welded web connection to transfer bending moment at the connection, thereby reducing the stress on the beam flange.

• **Specimens 3, 5, 6, 7 & 8: Reinforced connections.** The beam flanges were reinforced with either cover plates (Specimens 3, 5, 7 & 8) or with vertical "ribs" (Specimen 6). In each case, both the beam flanges and the reinforcement were welded to the face of the column. The intent of these connections was to significantly reduce the stress on the beam flange groove welds and surrounding base metal regions, and to move the location of the beam plastic hinge away from the face of the column. Connections with cover plates and ribs have shown promising performance in past tests. The design goal adopted for the reinforced connection was that the region of the connection at the face of the column should remain essentially elastic under the maximum bending moments and shear forces developed by the fully yielded and strain hardened beams. For the various reinforcement configurations tested, the section modulus of the reinforced cross-section was on the order of 1.6 to 2.0 times the section modulus of the unreinforced beam cross-section.

• **Specimen 9: Unconventional connection designed to avoid through-thickness loading of the column flange.** Forces in the beam flange were transferred to the column through "side-straps" connected to the outer edges of the beam flanges and to the outer edges of the column continuity plates.

The test specimens were constructed in two different fabrication shops in California. Welds that would normally be field welds were in actual construction done in the fabrication shop, but by field welders from erection companies in California. All welds were done by the self-shielded FCAW process. Three different FCAW electrodes (.120-in. diameter E70T-4, $^7/_{64}$-in. diameter E70T-7, and $^7/_{64}$-in. diameter E70TG-K2) were used for the flat position field groove welds for different test specimens.

The E70T-4 electrode was frequently use for this application in the past. Neither the E70T-4 nor the E70T-7 electrodes provide minimum specified notch toughness properties. The E70TG-K2 electrode was chosen to provide high toughness. This electrode provides minimum specified Charpy V-notch values of 20 ft.-lbs. at -20 degrees F.

Weld tabs were used for all groove welds, and then were removed by air carbon arc cutting after the welds were completed. The groove weld runoff areas enclosed by the weld tabs are where the welds are started and terminated. Consequently, these areas may contain defects and notches that may initiate a fracture. Removal of weld tabs is intended to provide a clean, notch free termination of the groove weld, reducing the opportunity for fracture initiation.

No back up bars were left in place at the bottom beam flange groove weld. Bottom flange back up bars, when used, were removed by air carbon cutting. Alternatively, for the connections with bottom flange cover plates, weld details were developed that did not require the use of a back up bar. Bottom flange back up bars were not permitted to remain in place for several reasons. First, it was believed that inspection of the bottom flange groove weld would be more reliable without a back up bar. Removal of the bars permits visual inspection of the weld root, and presumably permits more reliable ultrasonic inspection. Further, it was believed that the presence of the back up bar creates a notch at the root of the groove weld that may help initiate fracture.

For the top flange groove welds, back up bars were removed for some specimens but left in place for others. It was speculated that the top flange back up bar was not as detrimental as the bottom flange back up bar. At the top flange, the back up bar is not located at the extreme fiber of the cross-section, as it is for the bottom flange. Further, the observed damage patterns suggest the top flange was not as critical as the bottom flange.

Note that removal of weld tabs and back up bars was not common practice prior to the Northridge Earthquake. Experience with specimen fabrication indicates that removal of weld tabs and back up bars are extremely time consuming operations and would add significantly to the cost of a connection.

Test Results

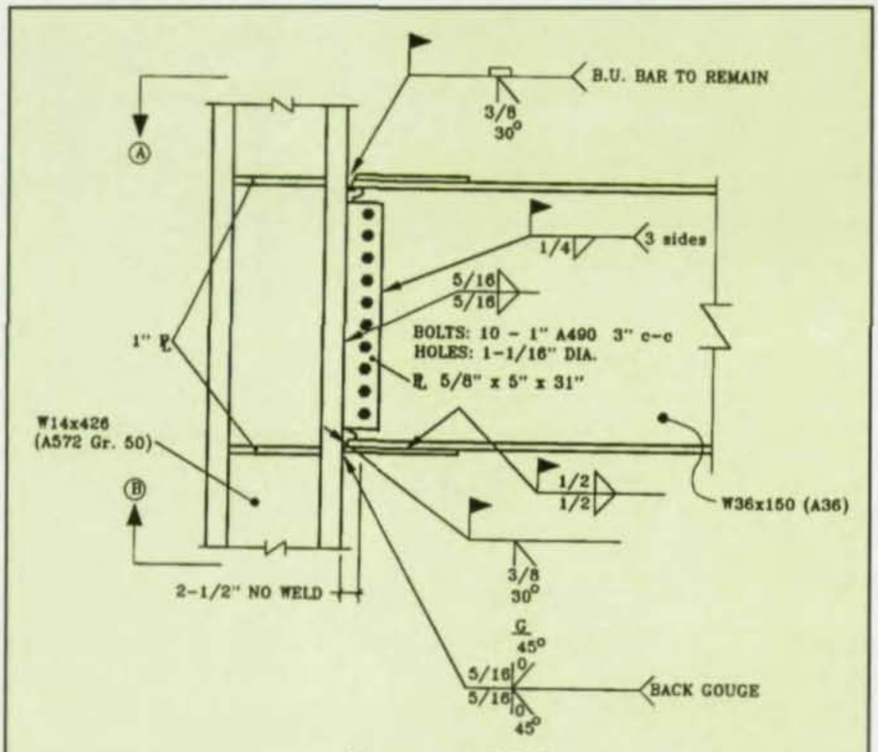
Both replicates of the welded flange-bolted web detail (Specimens 1A and 1B) as well as both replicates of the all-welded detail (Specimens 2A and 2B) showed unsatisfactory performance. All four connections experienced sudden fractures early in their inelastic loading histories, falling considerably short of achieving the target plastic rotations—despite very good welding workmanship, removal of the back up bars, and removal of the weld tabs. This suggests factors other than just poor workmanship contributed to the poor performance of these type of connections in the Northridge Earthquake.

It should also be noted that the poor performance of these four Specimens should not necessarily warrant condemnation of these connection details. These details may have shown better performance if, for example, a different welding electrode was used, or if continuity plates were used. Further testing in this area is needed before any definitive answers can be given.

Eight of the ten reinforced

Summary Of Test Specimen Performance

Spec.	Brief Description Of Failure	Max. Beam Plastic Rot.	Total Dissipated Energy (kip-in.)	Overall Assessment
1A	fracture at top flange groove weld, near weld-column interface	.005 rad.	680	very poor
1B	fracture at bottom flange of groove weld, near weld-column interface	.005 rad.	1600	poor
2A	fracture at top flange groove weld, primarily within weld metal	.0025 rad.	300	very poor
2B	fracture at top flange groove weld, primarily within weld metal	.009 rad.	2600	poor
3A	fracture at top flange and cover plate groove weld, near weld-column interface	.015 rad.	3750	poor
3B	gradual deterioration in strength due to beam local buckling, followed by gradual tearing of beam bottom flange at end of cover plate	.025 rad.	16,800	very good
5A	gradual deterioration in strength due to beam local buckling, followed by gradual tearing of beam bottom flange at end of cover plate	.025 rad.	19,700	very good
5B	brittle fracture at beam's bottom flange connection to column; fracture within column flange material ("divot" removed from column flange)	.005 rad.	1600	poor
6A	gradual deterioration in strength due to beam local buckling, followed by gradual tearing of beam bottom flange at end of ribs	.025 rad.	19,250	very good
6B	gradual deterioration in strength due to beam local buckling, followed by gradual tearing of beam bottom flange at end ofribs	.030 rad.	22,250	very good
7A	gradual deterioration in strength due to beam local buckling, and due to gradual fracture of fillet welds connecting cover plates to beam flanges; test terminated to avoid damage to test equipment; specimen still had substantial strength at end of test	.035 rad.	25,550	excellent
7B	same as 7A	.050 rad	25,500	excellent
8A	same as 7A	.035 rad.	26,300	excellent
8B	same as 7A	.035 rad.	25,800	excellent
9A	fracture of the side straps connecting the beam's top flange to the column	.015 rad.	7,600	fair
9B	gradual fracture along the edge of top and bottom flanges at connection to side straps	.015 rad.	7,300	fair



Specimens 7 & 8

connections showed excellent performance, typically sustaining on the order of 20 inelastic loading cycles, with maximum beam plastic rotations in the range of plus/minus 0.025 to plus/minus 0.035. For Specimens 7A, 7B, 8A and 8B, the test was terminated after the specimen had achieved very large plastic rotations in order to avoid damaging the testing apparatus.

Two of the connections reinforced with cover plates showed unsatisfactory performance, experiencing brittle failures at low levels of plastic rotation. One of these, Specimen 3A, failed by a sudden fracture at the top flange/cover plate groove weld to the column, with fracture occurring near the weld-column interface. As before, the fracture surface showed no evidence of workmanship errors. Specimen 3B, in contrast, showed excellent performance. The only difference between the two specimens was the choice of weld process variables for the E70T-4 electrode. The comparison of the two specimens emphasizes the importance of the Welding Procedure Specification. It is clearly the intent of AWS D1.1 that welding should be accomplished in accordance with a written WPS. These tests indicate that the development of a proper written WPS and its rigorous enforcement have a significant impact on the success or failure of a connection.

Specimen 5B also exhibited unsatisfactory performance. Failure occurred by a sudden fracture at the bottom beam flange connection to the column. The fracture appeared to be almost completely within the column flange material, and the cause of the failure is still under investigation. The nature of the failure, however, suggests that through thickness properties of the column flange may have played a role in the failure.

Conclusions

• The unsatisfactory performance of Specimens 1 and 2 suggests that improving welding workmanship, by itself, may not assure satisfactory performance.

• The tests of the reinforced connections indicate that it is possible to design and construct connections that will develop the full flexural strength of the beam, and permit the development of large plastic rotations under cyclic load.

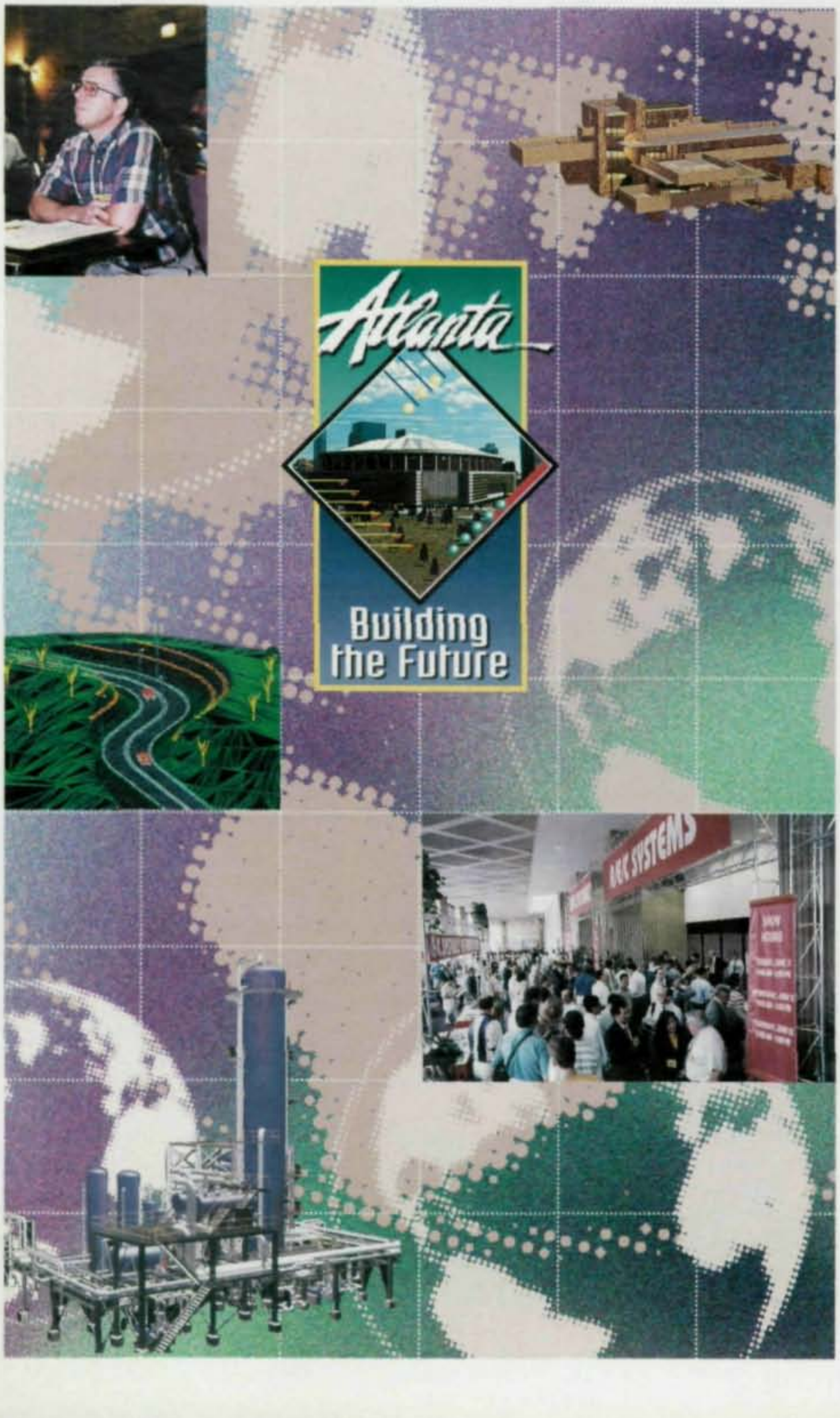
• The test results emphasized the importance of enforcing a written Welding Procedure Specification. The use of improper welding process variables can lead to deficiencies (e.g. low toughness) that are undetectable by ultrasonic testing or other common NDT techniques. Inspection and enforcement of the WPS at the time of welding is needed.

Some of the issues that require further investigation include the required level of toughness of the weld metal and base metal, the effect of the continuity plates, the influence of the top flange back up bar, and the role of column flange through thickness properties.

Michael D. Engelhardt, Ph.D., is an associate professor of civil engineering at the University of Texas at Austin; Thomas A. Sabol, S.E., is president of Englekirk & Sabol Consulting Engineers, Los Angeles; Riyad S. Aboutaha, Ph.D., is an assistant professor of civil engineering at Georgia Insitute of Technology; and Karl H. Frank, Ph.D., is a professor of civil engineering at The University of Texas at Austin. *This article was condensed from a paper delivered at the 1995 National Steel Construction Conference. Primary funding for the study was provided by AISC and the J. Paul Getty Trust. Additional funding/donation of materials or services were provided by PDM Strocal, Inc., the Herrick Corp., The Lincoln Electric Company, Twining Laboratories of Southern California, British Steel Inc., Nucor-Yamato Steel Co., TradeARBED Steel, Gayle Manufacturing, P.V. Banavalker, and the National Science Foundation Young Investigator Award (Grant BCS-9358186).*

A/E/C SYSTEMS® '95 & EDM℠ '95

for AEC • FM • GIS • EDM • CAD/CAM



The World Class Technology Event for Design and Construction

Conference June 5-8, 1995
Exhibit June 6-8, 1995
Atlanta, Georgia USA

Attend the world's largest CAD and computer show for design and construction professionals — learn how to use technology to
- produce quality projects
- on-time and within budget

A/E/C SYSTEMS '95 and EDM '95 plus the following concurrent events:
- Intellimap
- Facilities
- Virtual/Design
- M/cad Expo
- Autodesk Expo
- Bentley MicroStation Mall
- ESRI World
- IBM Technology Center

For complete information call 1-800/342-5718 (24-hour a day fax back service) or 203/665-0153 or fax 203/666-4782.

CALL
1•800•342•5718
TODAY!

STEEL MOMENT-RESISTING FRAMES AFTER NORTHRIDGE

Statistics on Northridge damage point to the need for probabilistic approaches to evaluation and design

Shown above is an example of weld and column flange fracture. The crack starts in the weld (lower right) and pulls out a divot from the column flange. Photo courtesy of David L. Norris, Western Inspection & Testing.

By David Bonowitz, S.E. and Nabih Youssef, S.E.

A SURVEY OF NORTHRIDGE EARTHQUAKE DAMAGE IS BEGINNING TO YIELD USEFUL CONCLUSIONS regarding steel moment-resisting frame (MRF) connections. Two patterns of damage, one quality-based and one demand-based, are emerging from the data. These patterns highlight the importance of design reliability, for which probabilistic approaches may be appropriate and even advantageous. Such approaches would account for documented variability in connection capacity.

One principle of seismic design is that any structural elements that "fail" in an earthquake should do so safely and reliably; i.e., they should maintain their strength and prevent collapse. Thus, the design philosophy behind steel moment-resisting frames (MRFs) allows for inelastic behavior in large earthquakes, but only in the ductile elements and specifically not in welded connections.

The Uniform Building Code (UBC), until last year, allowed a prescribed connection with full penetration groove-welded flanges and bolted and/or welded beam webs.

The standard connection, among other Code provisions for steel MRFs, is based on tests dating back to the 1960s. Typically, these tests involved elastic and inelastic cyclic loads applied to cantilevered wide flange sections. They did not, however, account for some conditions seen in actual buildings, such as the effects of column axial loads, the effects of composite concrete floor slabs, and the effects of true dynamic loads with high strain rates. Each of these is thought to have affected the patterns of observed Northridge damage.

Although most of the historical test programs were able to demonstrate the theoretical inelastic capacity of steel MRFs, published results reveal that post-yield behavior was not reliable. Typically, of eight or 10 specimens in a test program, one or two would fail prematurely. Dismissed then as aberrant behavior, the poor quality, brittle weld fractures and variable rotation capacity observed in these specimens appear in hindsight almost to have predicted Northridge damage. At the very least, they confirm that reliability in MRF connections is at least as great a concern as theoretical strength.

PRE-NORTHRIDGE DESIGN

In September 1994, the prescribed connection was removed from the UBC and replaced with a requirement to demonstrate connection capacity by test or by conservative calculations. Although specific features of the prescribed connection, which had been widely used in California for many years prior to its codification in 1988, have not been linked conclusively to observed damage, its removal from the Code should still be seen as a positive step. As Northridge damage was discussed and debated in late 1994, it became clear that most engineers knew very little about the history, metallurgy, construction and inspection of their full-penetration welds. This was certainly due in part to the availability of the prescribed detail. Removal of the detail may remove some of our institutionalized ignorance as well.

In particular, some engineers consider the fully-welded beam flanges to be fundamentally flawed. They argue that restrained steel in the standard detail can never reliably develop its plastic strength under impulsive earthquake loading.

Other pre-Northridge practices now under scrutiny include:

•Minimal structural redundancy. For example, the use of only two or three single-bay frames in each direction. Though economical, these frames use heavier column sections that may be prone to flange tearing; they have very low axial compressive stresses, which may lead to states of bi- or tri-axial tension; and they lose a greater portion of their strength when a single weld fractures.

•Backing bars left in place. The small vertical gap between the backing bar and the column flange can simulate a notch at the weld root, possibly leading to stress concentrations and fractures emanating from the root pass, a part of the weld that is sometimes prone to poor quality. Further, the presence of the backing bar can obscure UT readings and hinder quality control during construction. One solution is to routinely remove the backing bar after welding, backgouge the weld root pass, and refill the weld from the underside with a reinforcing fillet.

•Lax documentation and quality control. This regards all aspects of design and construction, from specifications of materials and procedures to certification of welders and inspectors. It is particularly important where traditional practices, such as the use of end dams, are at odds with project standards and where newer techniques, such as Flux-Core Welding, may be unfamiliar to project personnel.

While none of these practices has been shown conclusively to have caused the observed damage, all are thought to have helped make the damage more likely and more critical.

Shown above is a fracture in the heat-affected zone at the weld-colum face.

Northridge Earthquake Performance

An ongoing survey, funded by NIST and the SAC Joint Venture, of steel MRFs inspected since the Northridge earthquake has collected data from 24 engineering firms on 89 inspected buildings. The sample may be unrepresentative of the total MRF population because inspections so far have been voluntary, often motivated by the presence of non-structural damage and limited by budget and access constraints. In late February, the Los Angeles City Council passed an ordinance mandating limited inspection of MRF connections.

Contributing engineers completed survey forms with sections on building design criteria, detailing, configuration, earthquake performance, scope of post-earthquake inspection (typically visual and UT), and observed damage. The form defined six categories of damage based on the connection part most affected: Weld (at top and bottom of beam); Girder (top and bottom); Column Flange (top and bottom); Shear Connection; Panel Zone (continuity plates and double plate welds); and Column Web.

Data collected on a building-by-building basis would not have allowed studies of damage by frame direction or floor level. But due to schedule and budget constraints, data could not be collected for individual connections either. As a compromise, we collected damage data for each inspected "floor-frame," that is, each set of connections in a single frame at a single floor.

The survey buildings represent a variety of ages, sizes, frame configurations, degrees of regularity and redundancy, etc. If the survey has a typical building, it is a three- or four-story office building from the early 1980s, with a 20,000 to 30,000 sq. ft. floor plate and plan setbacks at the upper stories. It has two-to-four frames in each direction and two-to-four bays in each frame. Geographically, the surveyed buildings lie mostly in areas of concentrated commercial development. Surveyed buildings with serious MRF damage lie as far south as Santa Monica and West L.A., as far north as Santa Clarita, and along a seven-mile wide band north of the Santa Monica Mountains running 25 miles from Simi Valley in the west to Glendale in the east.

Survey statistics (as of February 1995):

Buildings	89
Investigated frames	590
Investigated floor-frames	2066
Visually inspected connections	8675
UT'd Connections	7812

Most common damage class (as percent of all inspected floor-frames)

Bottom weld	50%
Top weld	18%
Column flange at bottom	15%

The most serious damage, fracture through the column flange into the column web, was reported in 92 floor-frames (4%) in 21 buildings.

Note that fractures along the fusion zone between the weld and the column flange are generally counted as weld damage. Also, it is important to note that about half of all reported weld "damage" was reported as type W1, defined on the survey form only as "incipient root cracks detected by UT." Whether these rejectable conditions predated the earthquake is an open issue, although by now most engineers believe that at least some of the root cracks are not earthquake damage.

Overall, the distribution of damage is as follows:

Total Survey	89 Bldgs.	2066 Floor-Frames
No Damage	19%	43%
Weld Dam. Only	29%	40%
Bottom Dam. (weld or bare metal)	81%	54%

Considering that half of all weld damage was "incipient root cracking" only, it appears that about two-thirds of all inspected floor-frames (43% + half of 40% = 63%) had nothing more than root cracks.

Damage Correlation

Because the floor-frame data so far does not include estimates of structural demand, it is mostly a catalogue of observed damage types and distributions. Correlation studies of damage level versus site and design characteristics confirm only two clear patterns:

In the San Fernando Valley, frames oriented in the north-south direction had significantly more damage than those running east-west.

In three- to five-story buildings, more damage was found at lower floors (no pattern of damage vs. floor level was found for midrise and highrise buildings).

Limited as they are, these two correlations are still useful in guiding further inspection programs. They also suggest a predictable relationship between damage and demand, discussed below.

Despite the conjectures of some engineers, no statistical correlations have been found between observed damage and building size, building age, nominal material strength, structural regularity, structural redundancy, the number of bays per frame, frame dimensions, or member sizes. Planned studies with ground motion and analysis results may show that some design conditions are more critical than others above a certain threshhold demand, but so far it cannot be shown that some frames are more susceptible to damage due to their configuration alone.

In addition to the floor-frame

Shown above is another example of lamellar flange tearing (note use of end dam on weld, which is not supposed to be used). Photo courtesy of David L. Norris, Western Inspection & Testing.

data, we also are compiling and analyzing data from about 2,000 connections in eight buildings analyzed as SAC case studies. For each connection, we know the building and connection configuration, the observed damage, and the beam end moments and inter-story drifts from elastic analyses with recorded or synthesized Northridge ground motions. Preliminary results are encouraging, suggesting a correlation between damage and elastic demand. But, even if additional data support this trend, any correlation will almost certainly be probabilistic, not deterministic. In other words, we may be able to say that for a given demand-capacity ratio there is a certain likelihood that a connection will be damaged, but we will not be able to derive a "fracture strength" for a given detail.

This probabilistic approach is necessary because of the variable material strength, construction quality, and loading of a given MRF connection, as demonstrated by the history of lab testing. In fact, two global "damage" patterns are emerging:

• A largely random pattern of rejectable weld conditions caused by poor quality control during construction and more than likely pre-earthquake.

• Superimposed on the first pattern, a set of serious fractures statistically correlated to local stresses.

The first pattern, perhaps tied to weld quality, helps explain the observed scatter of weld root "damage" as well as the unlikely cases of buildings where most inspected welds had rejectable defects but none was cracked through. The remaining question is whether the two patterns are linked. That is, does poor initial quality increase the chance of serious damage?

This question will become central as more buildings are inspected and more rejectable weld discontinuities are found. If the defects are original, will insurers still cover the costs of inspection and repair? If ordinances mandate repair of damage, do they also apply to conditions once overlooked or even accepted? And if some of the cracks are original, what does it mean that they rode out the earthquake without fracturing completely? As with California's unreinforced masonry buildings, the MRF issues are more than technical, and their resolution will demand cooperative input from all potential stakeholders. But unlike UMBs, we still design and build steel MRFs with high expectations.

The second pattern shows up when cumulative distributions of damaged and undamaged connectins are plotted against elastic demand-capacity ratios (DCRs). Overall, the connections with visible base-metal fracture tend to have a higher proportion of damage. Although no critical fracture stress is identified and damage to specific connections cannot be predicted, these patterns confirm the value of analysis in setting the scope of post-earthquake inspection for large buildings.

Mitigation

SAC Advisory No. 3 provides practical recommendations for dealing with new and existing MRFs. For both, the first step is selecting appropriate performance goals, and here it is essential to distinguish between life safety hazard mitigation and damage or economic loss mitigation. Although there is still a range of opinion on the subject, most engineers believe that existing steel MRFs, if repaired where necessary, are capable of providing substantial life safety. And because life safety is the first priority of building officials and code writers, we cannot expect forthcoming guidelines or mandates to address damage control specifically. But considering the costs in business downtime from the Northridge earthquake, and considering that modern steel MRF tenants are frequently businesses with national and global interests, isn't damage mitigation also a reasonable goal? These owners and tenants may find a yellow-tag unacceptable even if they can walk out of their building unharmed.

If life safety is the only goal, then the challenge is to determine how many and which MRF connections can fail in the next earthquake without collapse. If damage control is important, then the issue is how to upgrade or design the frames to limit the scope of post-earthquake inspection and repair. In either case, design decisions can benefit from a probabilistic approach. Given the uncertainties of MRF connection design, a deterministic approach, which requires reliable, uniform behavior from each connection, may be impractical and excessively conservative. A probabilistic approach, on the other hand, would acknowledge that some connections simply will not perform as well.

A truly probabilistic approach, though not yet well defined, would rely on principles of statistics, reliability analysis, and performance-based design.

Probabilistic methods are already used to define earthquake demands in terms of specified risks, for example, the type of shaking expected to occur with a 10% probability in a 50-year building lifetime.

SETTING THE SCOPE

How many and which connections should be inspected after an earthquake? Fifteen percent? Two per frame per floor? As many as possible up to, say, \$25,000? Do tall buildings warrant more or less inspections than lowrises? Should highly redundant systems get a break? A rational life-safety approach begins by asking: "How much damage does it take to impair the life-safety capacity of the structure?" and follows with "How much inspection does it take to rule out such damage." The first answer depends on the redundancy and configuration of the MRF system. The second answer can only come from probability theory. It turns out that the certainty of finding randomly scattered damage/defects by inspecting a fixed percentage of connections depends heavily on the number of connections in the building.

For example, consider a recently shaken building that may have hidden damage among its 300 connections. Suppose that the owner's engineer, accounting for redundancy and frame configuration, determines that damage to 10% of the connections could impair the building. If they inspect 24 connections (only 8% of the total) and find no damage, then probability theory says they can be *more* *than 90% sure* that the critical damage level of 10% has not been reached. However, if the building had only 100 connections, a similar 8% inspection would be *less than 60% certain* that damage was less than critical.

Consideration of sample size also is essential because critical subsets of connections, such as those in the north-south direction or those in a single floor, must be treated seaparately to assure that local collapse modes are ruled out. And because subsets of connections may be small, the required percentage of inspection may be high.

These numbers assume randomly scattered damage and random inspection. They apply mostly to the first pattern discussed above in which damage or defects are related to initial construction quality. If ongoing research reveals a correlation between damage and analytical demand or floor level, for example, then this would of course affect the inspection scope. Following the theory of two independent damage patterns, one quality-based and one demand-based, a combination of random and focused inspection seems

Shown above is a crack through the column flange (bottom left) continuing into and ahlf way across the column web. Photo courtesy of Brandow & Johnston Associates rational. Alternatively, an engineer could propose a random program with a subset of connections identified by analysis.

Analyzing Frames

Analysis is traditionally an all-or-nothing deterministic exercise: For design in the elastic range, either a connection is overstressed or it isn't; and for non-linear studies, post-yield behavior is only as accurate as your hysteric model. Semi-rigid and fracture elements allow better representation of actual behavior, but a single MRF model still cannot capture the potential variability observed both in historical lab tests and post-Northridge inspections.

Alternatively, a simple probabilistic approach to evaluation or design of MRFs could involve a series of models, each with a certain number of connections discounted. As time-history seismic analysis considers one model with a suite of appropriate ground motions, probabilistic MRF analysis could consider a suite of damage scenarios subjected to one or more design basis load patterns. Such an approach would probably be most useful for checking a stability limit state.

The damage scenarios would reflect observed damage patterns, both quality-based and demand-based. For example, the NIST/SAC survey statistics show that less than 40% of all floor-frames sustained significant damage in one or more connections. Less than 25% of the analyzed case-study connections were visibly damaged. This suggests at least a starting point for simple probabilistic analysis: model 30-35% of a frame's connections as pins or semi-rigid elements. The scenarios would of course be tempered by any demand-based damage correlations yet to be confirmed, such as an increased likelihood of damage at certain floor levels or for higher elastic stresses. Similar approaches could be used in non-linear analysis to vary the hysteretic properties or fracture level of inelastic elements.

Testing New Details

Proposed details for repair, upgrade or new MRF construction are currently being tested. The probabilistic approach recognizes that actual performance of these improved details will rely as much on the quality of materials, construction, and inspection as on stresses and strains. Therefore, it is essential that these tests establish not only connection strength but connection reliability. The test programs should consider enough "identical" specimens to yield useful measures of the likelihood of different failure modes, including brittle fractures. In particular, researchers should consider how mitigation of one failure mode may increase the probability of another, for example, how beam strengthening could lead to higher ductility demands on the column and panel zone or how weld strengthening could force fractures out of the weld and into the column flange.

Nabih Youssef, S.E.is with the structural engineering firm of Nabih Youssef and Associates, Los Angeles. David Bonowitz, S.E., is based in San Francisco and is working with NYA on the SAC Survey Project. This article is based on a paper to be delivered at the National Steel Construction Conference. The first phase of the research discussed in the article was sponsored by the National Institute of Standards and Technology (NIST). John L. Gross, Ph.D., coordinated and supervised the work for NIST. Further data collection and processing was performed by Nabih Youssef & Associates under contract to the SAC Joint Venture (SAC), a partnership of the Structural Engineers Association of California, the Applied Technology Council, and California Universities for Research in Earthquake Engineering. Funding for the SAC work was provided by the Federal Emergency Management Agency (FEMA). SAC and FEMA do not endorse any findings or conclusions and are not responsible for any losses sustained as a result of the use of information or guidance contained in this publication.

AMERICAN GALVANIZERS ASSOCIATION

12200 E. Iliff Ave., Suite 204
Aurora, CO 80014
Ph: 303/750-2900
Fax: 303/750-2909
Contact: David G. Bueche
The AGA represents the galvanizing industry in North America. It serves as a technical and information resource to those engaged in the formation and design of products to be galvanized. AGA services include disseminating of technical and product application information through publications, videotapes and seminars.
NSCC Booth # TBA
For more information, Circle #103

APPLIED BOLTING TECHNOLOGY

P.O. Box 255
Ludlow, VT 05149
Ph: 802/228-7390
Fax: 802/228-7204
Manufactures a full-line of Direct Tension Indicators (DTIs) to ASTM F959-94. DTIs are a simple mechanical product that can make the installation and inspection of A325 or A490 bolts much easier by eliminating the need for torque wrench inspection on buildings, bridges and industrial structures.
NSCC Booth # 511
For more information, Circle # 104

ARKANSAS STEEL PROCESSING, INC.

P.O. Box 129
Armorel, AR 72310
Ph: 501/762-1000
Fax: 501/762-1411
Contact: Bob Bronson
Offers steel processing services to structural fabricators and steel service centers. Arkansas Steel can split tees from wide flange beams, camber beams, mill columns, and offers specialized processing. Also offers fully processed floor and perimeter beams, girders, and columns; as well as cutting, drilling, coping, camber, and bolted connections. The company is located adjacent to Nucor-Yamato's wide flange mill near Blytheville.
New this year: Outside crane runway to facilitate handling of material; plant addition for extra processing space; and upgraded sawing capabilities.
NSCC Booth # 528
For more information, Circle #125

THE ATLAS BOLT & SCREW COMPANY

1628 Troy Road
Ashland, OH 44805
Ph: 800/321-6846
Fax: 419/289-2564
Contact: Carolyn Gongwer
Manufactures a full line of fasteners and closures for the metal construction industry: Ultra Z self drilling & self-tapping fasteners in carbon, 410 & 18-8 stainless steel; Ultimate zinc die-cast and Colormate nylon molded long-life heads; Woodfast and Woodtite metal-to-wood, insulation & Con-Mate masonry fasteners; Sealing washers; Flasher pipe flashing; and Spectrum computer color matching.
NSCC Booth # 308
For more information, Circle # 122

BARESEL CORPORATION

300 S. Harbor Blvd., Ste. 500
Anaheim, CA 92805
Ph: 714/776-3200
Fax: 714/776-1255
Contact: Guenther Baresel
Produce the ComuSTEEL detailing software system.
NSCC Booth # 517
For more information, Circle # 131

BEHRINGER SAWS INC.

730 Hemlock Road
Morgantown, PA 19543
Ph: 610/286-9777
Fax: 610/286-9699
Contact: Joseph A. Dick
Company will feature it's HBP-360/704G semi-automatic horizontal miter bandsaw with motorized miter setting via N.C. preselect with plus or minus 1 degree of accuracy at this year's NSCC. In addition, information will be available on the company's complete line of saws.
NSCC Booth # 224,226
For more information, Circle # 129

BOYLES GALVANIZING

625 W. Hurst Boulevard
Hurst, TX 76053
Ph: 817/268-2414
Fax: 817/282-7793
Contact: John Blackburn
Offers full range of hot dip galvanizing of steel.
New this year: "Galv-Plus" is a new process that provides a smoother, brighter finish for galvanized steel and is offered exclusively by Boyles Galvanizing.
NSCC Booth # 524
For more information, Circle # 105

BULL MOOSE TUBE

1819 Clarkson Road
Chesterfield, MO 63107
Ph: 314/537-2600 ext. 262
Fax: 314/536-1225
Contact: Rolf Frederickson
Produces cold, direct formed ERW, HSLA hollow structural sections in sizes up to 12"x12"x$^5/_8$" square and 16"x8"x$^5/_8$" rectangular plus metric sizes up to 300x300x16mm and 400x200x16mm to major international certifications.
NSCC Booth # 334
For more information, Circle # 128

DCCAD

605 Royal York Road, Suite 201
Etobicoke, Ontario M8Y 4G5 CANADA
ph: 416/253-4848 or 800/776-4840
fax: 416/253-4315
Contact: Dave Cathra
SteelModeler 3D detailing software features interactive, on-screen modeling and customized macros for connections. Shop drawings and bills of material may be output automatically, to a variety of formats, at any time during detailing. DXF files may be imported, created and exported. MS-DOS, OS/2, AutoCAD and UNIX versions are available.
New this year: The program, which originated in Europe, has been completely "Americanized". Also, neutral file import and export is now available.
NSCC Booth #408, 411
For more information, Circle # 107

CANAM STEEL CORPORATION

P.O. Box C
Point of Rocks, MD 21777
Ph: 301/874-5141
Fax: 301/874-2646
Contact: Sam Blatchford
Offers: open web steel joists; specialty joists; trusses; Murox Wall System; design build services; and structural steel. The plant is AISC Category II Certified.
NSCC Booth # 103, 105, 202, 204
For more information, Circle # 64

CLEVELAND STEEL TOOL

474 E. 105th St.
Cleveland, OH 44108
Ph: 216/681-7400
Fax: 216/681-7009
Contact: Candy Peck
Produces punches, dies, punch stems, die holders, shear blades, coupling nuts and other related tooling.
NSCC Booth # 504
For more information, Circle # 67

COMEQ, INC.

P.O. Box 2193
Baltimore, MD 21203
Ph: 410/325-7900
Fax: 410/325-1025
Company distributes a wide range of metal fabricating

machinery, including: hydraulic upstroking and downstroking press brakes and hydraulic shears; hydraulic and mechanical shears and pipe and tube benders; hydraulic ironworkers and single-end punches; hydraulic press brakes; angle and plate bending rolls, beam benders, jogglers and welding positioners.
New this year: GEKA has introduced a CNC X-Y Positioning Table.
NSCC Booth # 205, 304
For more information, Circle # 68

Controlled Automation Inc.

P.O. Box 530
Bryant, AR 72022
Ph: 501/557-5109
Fax: 501/557-5618
Contact: Buck Rogers
Specialist in retrofitting existing fabrication equipment, including: Angle Lines (all brands); Beam Lines (all brands); Drill Lines (all brands); Plate Punches; and Plate Cutting Machines. Also sells new machinery, including: Angle Lines; Beam Lines; Plate Punches; Conveyor Systems; Portable Punch Systems; Marking Units; and Band Saws.
NSCC Booth #: 523
For more information, Circle # 106

Design Data

121 South 13th Street
Lincoln, NE 68508
Ph: 402/476-8278 or 800/443-0782
Fax: 402/476-8354
Contact: Doug Evans
This totally integrated software system encompasses the entire steel industry. Modules include: Detailing; Engineering, Analysis & Design; Estimating; and Production Control. The Detailing Module uses a solids modeling approach to create an exact replica of what is to be erected in the field. The Analysis and Design Module calculates forces, stresses and displacements and designs the structure taking into account all modes of failure and serviceability limits. All of the material can be transferred electronically or entered manually.
NSCC Booth # 109, 111, 113, 115
For more information, Circle # 32

Dogwood Technologies, Inc.

P.O. Box 52831
Knoxville, TN 37950-2831
Ph: 800/467-0096
Fax: 516/584-6032
Contact: Ron Cupp
DTI offers software for fabricators, detailers and engineers. Modules include: Fabrication Control; Shop Management; Detailing and Engineering Analysis Programs. Runs on IBM-compatible 386 and above.
New this year: Design Downloading module, which allows direct transfer of information from structural engineering software neutral file to DTI software. Also, Metric compatibility, X-windows graphics, and new CNC for various machinery.
NSCC Booth # 221, 320
For more information, Circle # 93

E.J.E. Industries, Inc.

287 Dewey Ave.
Washington, PA 15301
Ph: 412/228-8841 or 800/321-3955
Fax: 412/228-7668
Contact: Edward F. Easterday
New version of company's Structural Material Manager software system is now available.
New this year: V.5.1, the 11th release in the last decade, has a new Estimating Module that now incorporates a Master Price File feature. Also, the Plate-Nesting Module's reports now consume up to 50% less paper while drawing optimal cutting patterns, thanks to the addition of "autoscaling" logic. Finally, the system's Production-Control Module now displays a running tally of the current truck weight as items are entered on a Shipping Ticket.
NSCC Booth # 501
For more information, Circle # 46

EdisonWelding Institute (EWI)

1100 Kinnear Road
Columbus, OH 43212-1161
Ph: 614/485-8400
Fax: 614/486-9528
Contact: Harvey Castner
EWI offers a full range of services in materials joining technology for steel construction and manufacturing, including process selection and optimization, contract R&D, FEA/FEM, and failure analysis.
NSCC Booth # TBA
For more information, Circle # 108

Elco Industries/ Construction Products Division

P.O. Box 7009
5110 Falcon Road
Rockford, IL 6115-7009
Ph: 800/435-7213
Fax: 815/397-8986
Contact: Pat Monahan
Elco will display its Dril-Flex self drilling fasteners and its HangerMate threaded rod anchoring system at this year's NSCC.
NSCC Booth # 214
For more information, Circle # 109

Fab/Trol Systems Inc.

1655 West 11th Ave.
Eugene, OR 97402
Ph: 503/485-4719
Fax: 503/485-4302
Contact: Sue Landis
Offers an integrated suite of software programs designed to improve the management of structural, miscellaneous,

The Lincoln Electric Co. will be exhibiting its welding equipment in booth #309,311.

and general fabrication shops. Includes estimating, auto-laboring, nesting, inventory and remnant control, purchase order, production control, and shipping modules. Programs interface with American Contractors Accounting Software and with most major CAD programs, creating an integrated system. The software comes with an expandable database, creating the equivalent of a computerized AISC manual.
New this year: Fab/Trol V5 for Windows and DOS, plus Auto//Labor, Schedule/Mate and Bar/Coder modules. Also, American Contractors Accounting Software will be joining Fab/Trol at the NSCC this year.
NSCC Booth # 434
For more information, Circle # 94

Franklin Manufacturing

Highway 243, Industrial Prk.
Russellville, AL 35653
Ph: 205/332-0143
Fax: 205/332-0143
Contact: Mike Liveoak
Custom designs, manufactures, and installs machinery ranging from manual control fabrication equipment such as portable hydraulic punches, shears and plate rolls to complete CNC controlled machinery, including angle fabricators, plate fabricators, bus bar fabricators, drill

Mi-Jack will be exhibiting its material handling equipment at booth #424.

lines, drill centers, and structural fabricators complete with automatic loading and unloading accessories.
NSCC Booth # 310, 312
For more information, Circle # 126

HAYDON BOLTS

1181 Unity St.
Philadelphia, PA 19124
Ph: 215/537-8700
Fax: 215/537-5569
Contact: Rich Giusti
Manufactures anchor bolts, tie rods, plate anchors, U bolts, eye bolts and other structural specials up to 4" in diameter. Products are offered in ASTM A36, A449, A325, A588 and A490. Also, a complete line of A325 and A490 TC bolts and standard hex bolts (plain, hot dipped and mechanically galvanized) are offered.
NSCC Booth # 525
For more information, Circle # 101

HUCK INTERNATIONAL

8001 Imperial
P.O. Box 8117
Waco, TX 76714
Ph: 817/776-2000
Fax: 817/751-5259
Contact: Don Shell
Offers high-strength blind bolts, lock pin and collar fastening systems, and blind fasteners in various materials and finishes. Custom variations are available for specific performance requirements.
New this year: High-strength blind bolting for hollow-structural sections.
NSCC Booth # 327
For more information, Circle # 111

ITW BUILDEX

1349 W. Bryn Mawr Ave.
Itasca, IL 60143
Ph: 708/595-3500
Fax: 708/595-3569
Contact: Ralph Tenuta
Complete line of fastening systems to attach metal decking. Systems utilize self-drilling screws and powder activated pins.
NSCC Booth # 303
For more information, Circle # 110

INDIANA FASTENER — DIV. OF BIRMINGHAM FASTENER MANUF.

P.O. Box 151
Auburn, IN 46706
Ph: 800/595-2658
Fax: 219/927-8909
Contact: Peter F. Kasper
Offers a wide range of cold-forged and hot headed construction fasteners, including structural bolts in Grades A307, A325 and A490, as well as anchor bolts, sag rods, eye bolts, etc. Product line includes metric structural fasteners, twist-off bolts, and galvanized or Duracoat finished bolting. Company specializes in testing for bridge and highway use.
New this year: Metric structural bolting and Duracoat extended corrosion resistance coatings.
NSCC Booth # 526
For more information, Circle # 112

JANCY ENGINEERING

2735 Hickory Grove Road
Davenport, IA 52804
Ph: 319/391-1300
Fax: 319/391-2323

MODERN STEEL CONSTRUCTION Reader Service Card

Please type or print only

name

company name

address

city | state | zip code

( ) business telephone

( ) fax number

For free information on advertisers and other companies mentioned in this issue, circle the appropriate number below:

1	2	3	4	5	6	7	8	9	10	11	12
13	14	15	16	17	18	19	20	21	22	23	24
25	26	27	28	29	30	31	32	33	34	35	36
37	38	39	40	41	42	43	44	45	46	47	48
49	50	51	52	53	54	55	56	57	58	59	60
61	62	63	64	65	66	67	68	69	70	71	72
73	74	75	76	77	78	79	80	81	82	83	84
85	86	87	88	89	90	91	92	93	94	95	96
97	98	99	100	101	102	103	104	105	106	107	108
109	110	111	112	113	114	115	116	117	118	119	120
121	122	123	124	125	126	127	128	129	130	131	132

☐ Check this box to renew or start a **FREE*** subscription to *Modern Steel Construction* *This offer good only for practicing professionals within the U.S.

signature

1. Type of work
Please circle **all** that apply
a. Bridges
b. Office Buildings
c. Hospitals
d. Retail
e. Industrial
f. Schools
g. Multifamily
h. Institutional
i. Low-Rise
j. High-Rise

2. Your position
Please circle **ONLY** one
a. Owner/President
b. Chief Engineer/ Chief Architect/ Department Head
c. Staff Engineer/ Staff Architect/ Construction Manager
d. Other

3. Type of business
Please circle **ONLY** one
a. Structural engineer
b. Civil engineer
c. Structural steel fabricator
d. Other fabricator
e. Educator/libary
f. Architect
g. Building owner/developer
h. Steel mill
i. Erector
j. Steel product manufacturer
k. GC/CM
l. Student
m. Other

4. Reason for Inquiry
Please circle **ONLY** one
a. Information file
b. Immediate or Future Purchase

MAY

Use these cards to request information from manufacturers and other companies mentioned in this issue. Simply circle the numbers referenced in the advertisement or article, fill out the rest of the information requested on the card, and either mail it to: Modern Steel Construction, Creative Data Center, 650 South Clark St., Chicago, IL 60605-9960 or fax it to: **312/922-3165.**

Please type or print only

name

company name

address

city | state | zip code

( ) business telephone

( ) fax number

For free information on advertisers and other companies mentioned in this issue, circle the appropriate number below:

1	2	3	4	5	6	7	8	9	10	11	12
13	14	15	16	17	18	19	20	21	22	23	24
25	26	27	28	29	30	31	32	33	34	35	36
37	38	39	40	41	42	43	44	45	46	47	48
49	50	51	52	53	54	55	56	57	58	59	60
61	62	63	64	65	66	67	68	69	70	71	72
73	74	75	76	77	78	79	80	81	82	83	84
85	86	87	88	89	90	91	92	93	94	95	96
97	98	99	100	101	102	103	104	105	106	107	108
109	110	111	112	113	114	115	116	117	118	119	120
121	122	123	124	125	126	127	128	129	130	131	132

☐ Check this box to renew or start a **FREE*** subscription to *Modern Steel Construction* *This offer good only for practicing professionals within the U.S.

signature

1. Type of work
Please circle **all** that apply
a. Bridges
b. Office Buildings
c. Hospitals
d. Retail
e. Industrial
f. Schools
g. Multifamily
h. Institutional
i. Low-Rise
j. High-Rise

2. Your position
Please circle **ONLY** one
a. Owner/President
b. Chief Engineer/ Chief Architect/ Department Head
c. Staff Engineer/ Staff Architect/ Construction Manager
d. Other

3. Type of business
Please circle **ONLY** one
a. Structural engineer
b. Civil engineer
c. Structural steel fabricator
d. Other fabricator
e. Educator/libary
f. Architect
g. Building owner/developer
h. Steel mill
i. Erector
j. Steel product manufacturer
k. GC/CM
l. Student
m. Other

4. Reason for Inquiry
Please circle **ONLY** one
a. Information file
b. Immediate or Future Purchase

MAY

NO POSTAGE
NECESSARY
IF MAILED
IN THE
UNITED STATES

BUSINESS REPLY MAIL

FIRST CLASS MAIL PERMIT NO. 12522 CHICAGO, IL

POSTAGE WILL BE PAID BY ADDRESSEE

MODERN STEEL CONSTRUCTION

c/o CREATIVE DATA SERVICES
650 S CLARK ST
CHICAGO, IL 60605-9960

Use these cards to request information from manufacturers and other companies mentioned in this issue. Simply circle the numbers referenced in the advertisement or article, fill out the rest of the information requested on the card, and either mail it to: Modern Steel Construction, Creative Data Center, 650 South Clark St., Chicago, IL 60605-9960 or fax it to: **312/922-3165.**

NO POSTAGE
NECESSARY
IF MAILED
IN THE
UNITED STATES

BUSINESS REPLY MAIL

FIRST CLASS MAIL PERMIT NO. 12522 CHICAGO, IL

POSTAGE WILL BE PAID BY ADDRESSEE

MODERN STEEL CONSTRUCTION

c/o CREATIVE DATA SERVICES
650 S CLARK ST
CHICAGO, IL 60605-9960

MODERN STEEL CONSTRUCTION
Reader Service Card

Contact: Robert Debacker
Manufacturers of SLUGGER portable magnetic drilling machines and annular center-free cutting tools, capable of producing up to $2^1/_{16}$" diameter holes through 3" carbon steel in seconds. SLUGGER drills do not require pre-drilling or reaming and produce concentric burr-free holes in seconds. Drilling machines can be easily converted for use with twist drills to $^3/_4$" diameter. Cutters are available in both standard and metric was well as popular truck frame bolt sizes.
New this year: JM-5000 Drilling Machine and JM-150 Drilling Machine
NSCC Booth # 305
For more information, Circle # 72

J&M TURNER INC.

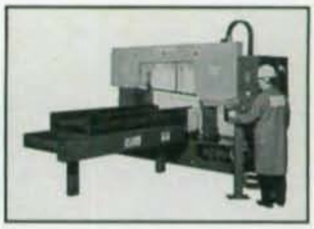
1310 Industrial Boulevard
Southampton, PA 18966
Ph: 800/525-7193
Fax: 215/953-1125
Manufacturers Direct Tension Indicators (DTIs), a direct reading load cell for structural bolts. When installed with a nut, bolt and washer assembly, DTIs positively identify that the bolt is properly tensioned. The company offers a complete line of DTIs in both inch and metric series, plain, mechanically galvanized and epoxy coated.
New this year: DTIs calibrated for use in tensioning bolts and studs used in the assembly of pipe and pressure vessel flanges.
NSCC Booth # 415
For more information, Circle # 69

KALTENBACH, INC.

P.O. Box 1629
Columbus, IN 47202
Ph: 812/342-4471
Fax: 812/342-2336
Contact: David Goetsch
Structural steel specialists in cutting (cold saw and band saw) and material transport. Cold saw applications are served with the "HDM" series, which specializes in the 24" to 48" range. Band saw applications are served with twin Colamne machines, which range from 14" to 40" capacity.
New this year: The HDM-1430, an automatic cold saw capable of full CNC structural cutting, fully automates material handling, length measure, material optimization, sorting, transfer and discharge applications.
NSCC Booth # 219
For more information, Circle # 113

LEJEUNE BOLT CO.

8330 W. 220th St.
Lakeville, MN55044
Ph: 800/872-2658



Peddinghaus will be demonstrating its fabrication equipment at booth

Fax: 612/469-5893
Contact: Brandt Dahlberg or Jason LeJeune
Offers a tension control bolting system with A325 (type 1& 3), A490 (type 1 & 3) and A325 (type 1, mechanically galvanized) bolts and associated wrenches for installation.
New this year: Full line of hex structural bolts from $^5/_8$" to $1^1/_4$".
NSCC Booth # 403, 405
For more information, Circle # 73

LINCOLN ELECTRIC CO.

22801 St. Clair Ave.
Cleveland, OH 44117
Ph: 216/481-8100
Fax: 216/486-1751
Contact: Dave Hutchens

Company will display a variety of power source welders and wire feeders at this year's NSCC. Included will be the Commander 400 engine-driven stick welder and a number of other engine drives as well as high productivity power source/wire feeder combinations. New this year: The Commander 400 is a reliable, industrial engine-driven DC stick welder with AC power. Also offers flexibility to pick and place with a lift hook and an easy fork-lift access.

NSCC Booth # 309, 311
For more information, Circle # 57

LOHR STRUCTURAL FASTENERS

P.O. Box 1387
Humble, TX 77032
Ph: 800/782-4544
Fax: 713/821-5216

Company will demonstrate the domestic high-strength rapid tension bolt system. The system provides structural joint integrity, higher installed tensions, while reducing installation and inspection costs.

NSCC Booth # 203, 302
For more information, Circle # 130

MNH-SMRF SYSTEMS

3151 Airway Ave.-N-1
Costa Mesa, CA 92626
Ph: 800/475-2077
Fax: 714/540-0319
Contact: Gary D. Myers

The company's new column and beam connection system for steel moment frames is available for new and retrofit construction. The connection eliminates full penetration welds and associated UT testing and inspection, as well as through thickness problems of column flange and restrained pre-stressed welds.

NSCC Booth # 404
For more information, Circle # 114

MID SOUTH BOLT & SCREW CO.

499 Cave Road
Nashville, TN 37210
Ph: 800/251-3520
Fax: 615/885-6542

Manufactures anchor bolts, plate & sleeve assemblies, tie rods, studs, swedge bolts, U-bolts, hex bolts and eye bolts. Offers structural bolts, nuts & washers in A325 and A490; also tension control bolts. Various grades and materials, including galvanized, are available.

NSCC Booth # 211
For more information, Circle # 74

MI-JACK PRODUCTS

3111 West 167th Street
Hazel Crest, IL 60429
Ph: 708/596-5200
Fax: 708/225-2312
Contact: Dan Gray or Dan Heraty

Offers self-propelled, rubber-tired gantry cranes able to pick and carry its capacity safely. Capacities range from 15 to 325 tons and dimensions are customized to individual applications.

NSCC Booth # 424
For more information, Circle # 115

NAPTECH INC.

851 S. Freeport Industrial Parkway
Clearfield, Ut 84015
Ph: 801/773-7300

Fax: 801/773-5185
Contact: Rick Renz or David Murphy
Specializes in structural steel hot induction bending for squares, rectangles, I-beams, and pipe.
NSCC Booth # 234
For more information, Circle # 116

N.C. Engineering Co.

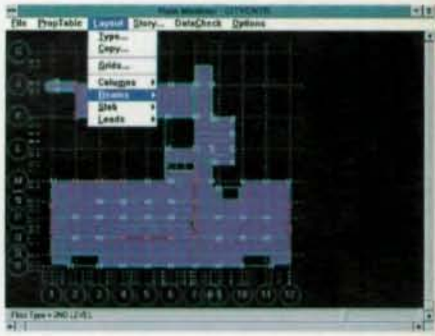
#228-4664 Lougheed Highway
Burnaby, British Columbia
V5C 5T5 CANADA
Ph: 604/293-0003
Fax: 604/293-0197
Contact: David Easingwood
Provides structural and miscellaneous steel detailing services. Company, which was established in 1966, has 2,500 hours/week production capacity (staff of 62).
NSCC Booth # 235
For more information, Circle # 117

Nucor-Yamato Steel

P.O. Box 1228
Blytheville, AR 72316
Ph: 501/762-5500 or 800/289-6977
Fax: 501/763-9107
Contact: Robert W. Johns
This joint venture steel company produces wide flange structural shapes from W6x15 to W40x397, H-piling sections from HP8x367 to HP14x117, selected channel and miscellaneous channel sections, and a variety of specialty sections. Available steel grades include ASTM A36, ASTM A572 Grades 50 & 60, ASTM A588 (and its equivalents, including ASTM A709), CSA 40.21 Grade 44W, ABS Grades AH32 & AH36 and other specialty grades.
NSCC Booth # 530, 532, 534
For more information, Circle #39

Peddinghaus Corp.

300 N. Washington
Bradley, IL 60915
Ph: 815/937-3800
Fax: 815/937-4003
Contact: Lyle Menke
This year's NSCC exhibits include:
•AFCPS-623-K Anglemaster, which features the proprietary Peddimat roller feed drive and measuring system that previously were not available on any CNC angle fabrication lines
•Twin Column Structural Steel Band Saw 38/18 with guillotine approach for maximized efficiency
•FPB Plate Punching and Air Plasma Cutting System (AFCPS 623K), which offers an economical solution for the automatic production of plate components that require punching, burning, marking and cutting .
NSCC Booth # 313, 315, 412, 414



Ram Analysis will be demonstrating its software at booth #215.

For more information, Circle # 118

Ram Analysis

5315 Avenida Encinas, Suite 220
Carlsbad, CA 92008
Ph: 619/431-3610
Fax: 619/431-5214
Contact: Gus Bergsma
RAMSTEEL is a special purpose structural engineering program for the analysis,

design and drafting of steel buildings. The program automates the process of designing steel beams, girders, joists, joist girders, columns and base plates. It allows the design engineer to build a model of the entire building graphically, with the program automatically computing the tributary loads for all members. Designs in either LRFD or ASD, and offers different alternatives for comparison. RAMSBEAM is a full featured single beam design program, excellent for designing one beam at a time. It analyzes or designs any loading for either steel or steel composite beams and girders. LRFD and ASD designs can be compared at the click of a button.
New this year: RAMSTEEL V4.1 includes direct importing of DXF CAD drawings. RAMSBEAM V2.0 includes graphical data verification, unbraced flange design check, and on-line help.
NSCC Booth # 215
For more information, Circle # 81

Research Engineers

22700 Savi Ranch
Yorba Linda, CA 92687
Ph: 714/974-2500
Fax: 714/974-4771
Contact: John Putnam
Company will be displaying its world-class structural engineering software at this year's NSCC. Included will be STAAD-III, AutoSTAAD and STAAD-Mate.
NSCC Booth # 402
For more information, Circle # 34

Romar Elite Automation Inc.

P.O. Box 3417
Tustin, CA 92681-3417
Ph: 714/569-1050
Fax: 714/569-1009
Contact: Peter J. Janes

Specializes in new and used steel fabrication equipment, including: beam coping machines; material handling systems; measuring devices for structural fabricators; column rotators; and web punches.
New this year: Tank turning rolls; positioners; manipulators; box column production line; welded beam line; tank welders; and other automated equipment.
NSCC Booth # 318
For more information, Circle # 119

SAWS International

1206 Kishwaukee St.
P.O. Box 3111
Rockford, IL 61106
Ph: 815/965-6900
Fax: 815/965-0761
Contact: Fan Vincent
Manufactures: HSS solid blades; segemental saw blades; friction saw blades; carbide tipped blades; bandsaw blades; and cutting fluid.
NSCC Booth # 505
For more information, Circle # 120

Softdesk, Inc.

7 Liberty Hill Road
Henniker, NH 03242
Ph: 603/428-3199
Fax: 603/428-7665
Contact: Christine Ek
The world's largest AutoCAD developer (more than 100,000 customers world-wide) offers a complete suite of integrated structural software. Softdesk products handle the complete structural process, including modeling, link to an industry standard analysis program, and automation of the generation of design/construction and fab-

rication shop drawings.
NSCC Booth # 208
For more information, Circle # 99

Southern Coatings

P.O. Box 160
Sumter, SC 29151
Ph: 803/775-6351
Fax: 803/775-7666
Contact: Skip Pendry
Offers a full-line of VOC compliant coatings for industrial use. All coatings include high solids and water borne coatings. Products include: Chemtec 608 Inorganic Zinc Rich; Dura-Pox 646 Epoxy Mastic High Build; Enviro-Guard Build Urethane; and VOC compliant dipping primers. New this year: Tint-based color systems for all major lines that afford an extensive color selection and fast shipment of special colors.
NSCC Booth # 209
For more information, Circle # 121

Steel Solutions Inc.

Rt. 3 Box 312A
Buckhannon, WV 26201
Ph: 304/472-2668
Fax: 304/472-3214
Contact: Ron Taylor
STRUCAD 2000 is a completely integrated management software for steel fabricators. Modules include: STRUCAD Detailing and STEEL 2000 Estimate, Mill Orders, Purchase Orders, Inventory and Production Control.
New this year: A 3D Modeling Solution.
NSCC Booth # 531, 533, 535
For more information, Circle #98

SteelCAD

831 North Irma Ave.
Orlando, FL 32803
Ph: 800/456-7875
Fax: 407/849-0202
Company will demonstrate the latest releases of 10 programs, ranging from detailing to CNC control, at this year's NSCC.
NSCC Booth # 419, 421, 518, 520
For more information, Circle #84

Structural Steel Systems

3763 Lower Saucon Road
Hellertown, PA 18055
Ph: 610/838-7338
Fax: 610/838-7338
Contact: Jan Hemp or Bud Panick
Specializes in buying, selling, leasing and brokering equipment typically found in structural and plate fabricating facilities. Offers technical sales personnel, service staff, remanufacturing of used equipment, retrofits and financing options.
NSCC Booth # 314
For more information, Circle # 127

Timberline Software

9600 S.W. Nimbus
Beaverton, OR 97008
Ph: 800/944-3609
Fax: 503/526-8049
The Precision Collection estimating package offers an easy-to-use spreadsheet format and a pre-built steel database. Crew capabilities, a dimensional audit trail and database security are also available. The software integrates with job cost, AutoCAD and scheduling programs.
NSCC Booth # 409, 411
For more information, Circle # 123

TradeARBED

825 Third Ave.
New York, NY 10022
Ph: 212/486-9890
Fax: 212/355-2159
Contact: J.C. Gerardy
ARBED manufactures unique, high-quality products, including:
•Tailor-made wide flange beams in both standard and jumbo sizes (from 24" to 36" up to 798 lbs./ft., as well as 40" and 44" deep beams)
•HISTAR quality steels (ASTM A913) in Grades 50, 60, 65 and 80; the material is ideal for gravity columns and trusses and has high-strength, good toughness and superior weldability
•HISTAR H bearing piles
•Steel sheet piling & steel wall program
•Tees, girder and crane rails
•KRYBAR (special ultra low temperature reinforcing bars)
NSCC Booth # 502
For more information, Circle # 35

Voss Engineering

6965 N. Hamlin Ave.
Lincolnwood, IL 60645
Ph: 708/673-8900
Fax: 708/673-1408
Contact: Glen Adams
Company fabricates and manufacturers structural bearing pads and bearing assemblies for bridges and commercial structures. Products featured at the NSCC will be: NEOSORB (AASHTO grade Neoprene pads); VOSSCO (Random Oriented Fiber Pads); SORB-TEX (preform fabric pads); and FIBERLAST (engineering ROF pads). In addition, information will be available on PTFE Expansion Bearings.
NSCC Booth # 125
For more information, Circle # 124

Vulcraft

P.O. Box 186
Grapeland, TX 75844
Ph: 409/687-4665
Fax: 409/687-4290
Contact: Randy Beard
Vulcraft, a division of Nucor Corp., will exhibit their full line of open web steel joists, joist girders, composite joists, and steel roof and floor decks. The company can manufacture a variety of special joists for super-longspan, scissor, bowstring and arched-chord truss applications.
NSCC Booth # 503, 532, 534
For more information, Circle # 37

Wasser High-Tech Coatings, Inc.

8041S. 228th St.
Kent, WA 98032
Ph: 206/850-2967
Fax: 206/850-3098
Contact: Bill Brinton, Jr.
Manufactures single-component moisture-cure urethane high-performance coatings. The product offers numerous advantages over two-part epoxy and urethane coatings and inorganic zinc coatings, including the ability to apply in very cold, damp or high-humidity conditions.
New this year: Zero VOC and less than 100 grams per liter VOC single-component moisture-cure urethane product line. Also, accelerated single-component moisture cure coatings that can be overcoated in less than 30 minutes.
NSCC Booth # 425, 427
For more information, Circle # 50

Yamazen Inc.

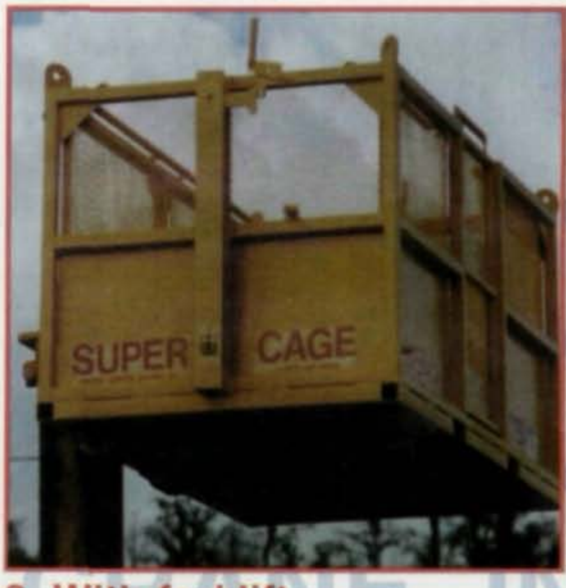
735 E. Remington Road
Schaumburg, IL 60173
Ph: 708/882-8800
Fax: 708/882-4270
Contact: Shin Takabe or Amy Komeda
Offers structural band saw machines and CNC structural drilling machines. The saws feature a patented amplifying valve feed system, unique three contact point mitering method, six degree saw bow angle and carbide roller ring back-up guides. The CNC drilling machines features an exclusive "fixed work piece-travelling spindles" method, which allows for faster hole location without repositioning heavy beams and results in greater accuracy.
NSCC Booth # 225, 324
For more information, Circle #80

THERE IS ONLY ONE DFA SUPERCAGE™ FOR THREE DIFFERENT APPLICATIONS

*Patent Pending

1. Attach directly to boom point

2. Suspend from crane hook with 4 or 5 part lifting bridle assembly



3. With forklift

Please circle # 86

For More Information:

UNITED STATES CRANE, INC.

P.O. Box 593290, Orlando, FL 32859   1-(800)-327-0300   1-(407)-859-6000   FAX 1-(407)-857-9146

SUPERCAGE PRODUCTS ARE CERTIFIED TO MEET OR EXCEED APPLICABLE OSHA, STATE AND FEDERAL REGULATIONS

STAAD3-2D

State-of-the-art 2D Structural Analysis and Design Software for $595

Research Engineers, Inc. is proud to announce the arrival of STAAD3-2D, the most comprehensive 2D Structural Analysis and Design software in today's industry. STAAD3-2D is the 2D version of RE's world famous structural software STAAD-III. Equipped with powerful Analysis/Design, brilliant GUI, easy Visualization/Verification, and flexible Report/Query facilities, STAAD3-2D represents the true state-of-the-art in modern structural engineering.

STAAD3-2D supports STEEL/CONCRETE/TIMBER design per AISC (ASD & LRFD), ACI and AITC codes. With a price tag of $595, it offers you the best value for all your 2D structural needs.

Research Engineers, Inc.
22700 Savi Ranch
Yorba Linda, CA 92687
Toll Free: 1-800-FOR-RESE
Fax: (714) 974-4771

*StruCAD*3D*

The Flagship Product for 3D Structural Analysis & Design

- Pop-Up & Pull-Down Menus
- Truly Interactive 3D Graphics Pre- and Postprocessing
- 3D AutoCAD Interface (in & out)
- AISC-ASD, AISC-LRFD, ACI-318
- Reinforced Concrete Design
- Wind & Gravity Load Generation
- Material Takeoff
- P-Delta and Thermal Analysis
- Pile-Soil Interaction
- Seismic & Machine Vibration
- Detailed Fatigue Analysis

Prices start at only $249

Zentech, Inc. 8582 Katy Freeway, #205, Houston, TX 77024
Tel (713) 984-9171 Fax (713) 984-9175

Structural & Miscellaneous Steel Detailing CAD Programs

AutoCAD parametric LISP programs for preparing structural AND misc. steel shop drawings. All programs have been updated for 1995 and new programs have been added. Anchor bolts to roof opening frames and "everything between". Imperial and metric versions with USA, Canadian and European steel sections. 45-DAY MONEY BACK GUARANTEE. Discounts up to 30%. Buy only what you need. Used in the field for 8 years. Programs written by a detailer with 39 years experience. Extremely "flexible" programs. FREE telephone support. No maintenance fee. Reasonable yearly update fee. EXCELLENT PROGRAMS at a REASONABLE PRICE. Call for a FREE demo disk, list of programs, and prices.

SSDCP
110 Shady Oak Circle, Florence, MS 39073
Tel: 601/845-2146 (fax same)

HEWLETT-PACKARD

Computers/Peripherals

A complete line of used and refurbished HP Equipment to fill all your computer needs. Laser printers, scanners, disk drives, plotters (Draftpro, Draftmaster & Designjet), PC's and 9000 series workstations are available for immediate delivery. Call our toll free number for additional information and pricing.

Ted Dasher & Associates
4117 2nd Avenue South
Birmingham, AL 35222
800-638-4833 fax (205) 591-1108

Images—3D

2D-3D Structural/Finite Element Analysis

- Easy to learn and use
- Shear & Moment Diagrams
- AISC Code Check
- Large Problems to 3,000 Joints
- Automesh Generation
- Static, Modal, Dynamic
- Enforced Displacements
- P-δ Analysis

Complete Static Package—Only $795

Celestial Software, 2150 Shattuck Ave., Suite 1200
Berkeley, CA 94704
Tel: 510-843-0977 / Fax: 510-848-9849
Fifty-node fully functional evaluation package for only $49.95

FOR SALE

NORTHERN COLORADO FRONT RANGE
STEEL FABRICATION BUSINESS
Fabricate components for Utility Companies
$1,000,000 plus gross per year.
Price $750,000 including real estate.
Call Larry or Bill
F&M Land Co. (970) 484-9088

HELP WANTED

STRUCTURAL STEEL ESTIMATOR

Large Midwest Structural Steel Fabricator has a requirement for a Structural Steel Estimator with a minimum of 10 years experience
Please mail your resume & salary requirements to:
Carter Steel and Fabricating Co.
P.O. Box 70
Bellefontaine, OH 43311-0070
Attn: Personnel Manager

Help Wanted

CHIEF DRAFTSMAN

Well established AISC Certified Category II medium size structural and miscellaneous fabricator needs well qualified department head. Reply in confidence via fax or mail to:
Stein Steel & Supply Company
P.O. Box 17907
Atlanta, GA 30316
Attn: Bert Stein, President
Fax: (404) 523-8415

METROSOFT STRUCTURES, INC. is looking for a motivated individual to join our growing team. Duties will include sales and pre-sales support of leading structural analysis software package ROBOT V6. Good working knowledge of competing products a plus. We offer a competitive salary and an excellent benefits package. Please send resumes to Metrosoft Structures, Inc., 332 Paterson Ave., E. Rutherford, N.J. 07073, Attn: Personnel, or you may fax it to us at (201) 438-7058.

Help Wanted — Draftspeople/Checkers

Multi-location structural steel fabricator located in the beautiful Ozarks of Missouri is seeking experienced structural steel draftspeople and checkers. Must be self motivated and want a challenging job.

AutoCAD-DCA or PDS cad experience a plus. We offer a competitive salary with benefits.

B&D Fabrication
2431 Stephens Blvd.
Joplin, MO 64801

Doing Steel, Inc.
2125 North Golden
Springfield, MO 65803

"California Structural Engineering Exam Review Manual"

Complete problem statements and solutions for the last 8 years of California Structural exam are presented (except the bridge problems). The main volume ($150) includes the 1990-1994 exam problems. These solutions are in a word processed format with extensive use of computer generated drawings for additional clarity. The 1987 thru 1989 solutions ($25 each) are also available as supplements. All solutions are per 1991 UBC, 1989 ACI and ASD manuals. Specific solutions and page numbers are printed at the sides of pages to provide ease and quickness for the user. To inquire about the technical contents, contact the editor, Ben Yousefi, S.E., at 619-236-7368. To order, call Bernie or Elisa at 619-236-7075.

Advertisers' Index