MODERN STEEL CONSTRUCTION

June 1995 $3.00

Richard Weingardt: The Image Behind The Design

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

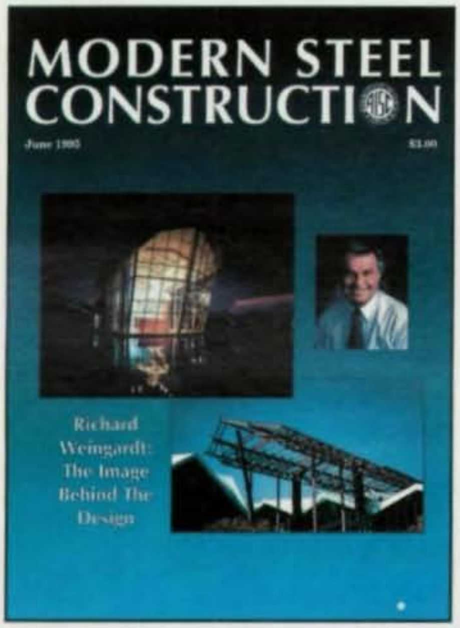
Note: In our Metric catalog "Steel Decks for Floors and Roofs", the tables on pages 33 and 35 are in the wrong place. Contact us for the needed corrections or contact us for a copy of the corrected publication.

NICHOLAS J. BOURAS, INC.
P.O. BOX 662, 475 SPRINGFIELD AVE.
SUMMIT, NEW JERSEY 07902-0662
(908) 277-1617

Please circle # 33

MODERN STEEL CONSTRUCTION

Volume 35, Number 6 **June 1995**



The expansion of the Cowboy Hall of Fame in Oklahoma garnered an ACEC Excellence Award for Richard Weingardt Associates, a firm known not only for its engineering prowess, but also for its marketing acumen

FEATURES

24 ESSENTIALS OF LRFD (PART 1 OF 3)
An overview of LRFD as found in Part 2 of the Manual of Steel Construction *(1994)*

30 THE IMAGE BEHIND THE DESIGN
Richard Weingardt's firm has designed more than 3,000 projects during the past 30 years—and he's not shy about telling the world

40 AN ALTERNATIVE TO JOIST GIRDERS
Talk of slow deliveries has prompted consideration of using rolled shapes in place of roof joist girders

DEPARTMENTS

6 EDITORIAL

9 STEEL INTERCHANGE
- *Fire wall construction*
- *Enlarging an existing footing*
- *Flexural design strength of a single angle*

12 STEEL NEWS
- *Letters to the editor*
- *Teaching about steel connections*
- *Work on connections garners T.R. Higgins Award*
- *AISC names new VP of engineering*
- *Metric guide for steel fabricators*
- *New price for LRFD on CD*
- *Environmental compliance workshops*
- *Focusing on practical steel design*

50 STEEL MARKETPLACE

Modern Steel Construction (Volume 35, Number 6). 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.

Beautiful Music

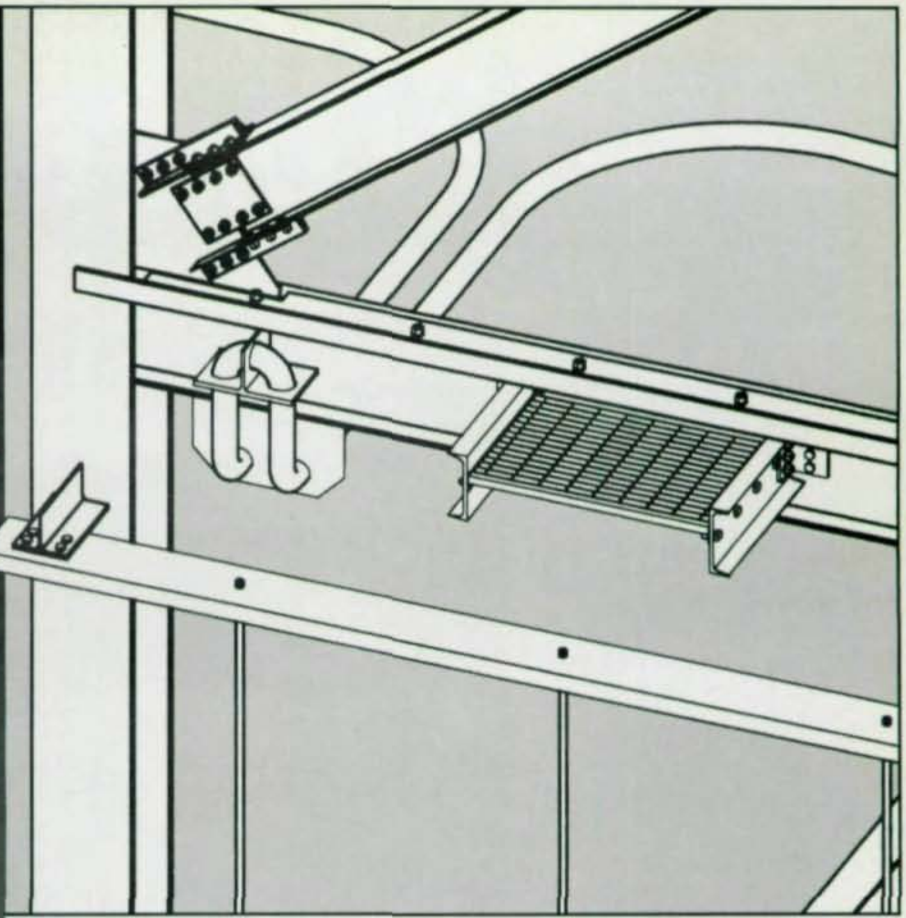
A FEW WEEKS AGO, A GOOD FRIEND HAD A COUPLE OF TICKETS TO THE CHICAGO SYMPHONY ORCHESTRA that he couldn't use and asked if I wanted them. I gladly accepted. I hadn't been to the symphony in a few years, and while I greatly enjoyed myself, I was struck by a difference in the way I approached the performance than I had back in my college days.

When I was in college more than a decade ago, I often took advantage of the free symphonic performances put on by Northwestern University's fine music school. Given the quality of my dorm room stereo, it was the only way to really hear fine music. In fact, even on the best of my friends' stereos, there was no comparison between the quality of a live performance and recorded music. But today, with the ubiquity of compact disc players, the best audio performances are in living rooms rather than in concert halls. Going to a concert, then, takes on an entirely new dimension. Today, you go to a concert to experience the music, to see the musicians, to partake of a public performance.

As with the performance of music, the design of structures has also shifted. A decade ago, few designers were making extensive use of computers. And for steel design, you had no choice but to use Allowable Stress Design. But today, engineering has moved to a great reliance on computers and steel design has moved to the more advanced and more reliable Load & Resistance Factor Design specification.

Just as the phonograph has given way to the CD player, ASD will soon give way to LRFD. To quote the AISC Board of Directors: "LRFD is a modern and technologically superior steel design specification." Still, despite LRFD's introduction nine years ago, some engineers are not familiar with its basic precepts. To help them out, beginning this month *MSC* is publishing a three-part series on the "Essentials of LRFD." This overview of LRFD, which starts on page 24, is taken from the LRFD Manual of Steel Construction (2nd edition) and is basically a primer on switching from ASD to LRFD. For even more information, AISC's upcoming seminar series will include a session on "LRFD for the Practicing Engineer" (a schedule appears on page 23).

Today, stereo albums are mostly found only in old collections. It won't be much longer before ASD manuals also are only a collector's item. **SM**




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

STEEL INTERCHANGE

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

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

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

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

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

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

For fire wall construction, building codes say the wallshall have sufficient stability to allow for collapse on either side of the wall without collapse of the wall. What is the optimum detail for this type of connection.

The sketch provided with the question submitted is applicable for a specific type of tied fire wall; non-loadbearing, constructed between two double-column lines, structural members on each side of the wall at the same elevation and primary framing members parallel to the wall. For a scenario such as this the recommendations of Factory Mutual Loss Prevention Data Book 1-22 are very specific and are as follows.

The anticipated horizontal component of the force resulting from the collapse of the structural frame on one side of the wall should be resisted by the remaining structure on the opposite side of the wall. This is accomplished through the use of through-wall ties. The ties are designed based on the horizontal pull "H" calculated from the formula provided in Recommendation #3 of the referenced FM Data Book, using an allowable stress of not more than 10 ksi. A detail of the recommended installation of the through-wall tie at each column line is shown in Figure 12 of the same FM Data Book. For the situation indicated by the sketch in question, it may be necessary to also install ties more often than every column line. In either case, enough slack should be provided in the tie connection to allow for normal building movement.

While the through-wall ties insure the continuity of the opposing framework at the fire wall, flexible masonry anchors should be provided at approximately 2 to 4 feet on center to brace the wall laterally (see Figure 13 of the referenced FM Data Book). It is important to note that enough slack should be provided in the anchors to compensate

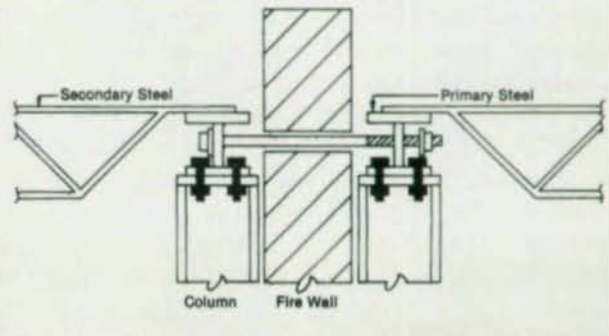


Figure 12: Through Wall Tie Primary Steel Parallel To Fire Wall

for the slack provided in the through-wall ties. This slack insures that the collapsing frame on the fire side of the wall will not pull on the wall before there is resistance provided from the frame on the unexposed side of the wall via the through-wall ties.

The Factory Mutual recommendations also include provisions for adequate separation between the double-column line and the fire wall to prevent damage to the unexposed structure during the initial stages of the fire.

D. Matthew Stuart, P.E.
The Stellar Group
Jacksonville, FL

What is the most efficient way to enlarge an existing footing, when new loading conditions are applied?

There is a good article dealing with this problem. It was published in the fourth quarter AISC Journal 1980 and was authored by Agrawal and Stafiej. The parameters required to solve the problem include ratios of the respective moments of inertia of the two column sections, ratios of axial loads applied at the top of the column to loads at the lower section, and ratios of the upper length to the lower length. Using these ratios, one then uses a chart which gives equivalent effective lengths factors for the composite column for six different end condition cases, pin-pin, fix-free (Steel Interchange question case), fix-pin, fix-slider, fix-fix, fix-pin, and pin-slider. From the determined effective length factors, the effective lengths of the upper and lower column sections are easily obtained for use in the Euler buckling formula.

James F. McCarthy
Folsom, CA

Is the method of determining the flexural design strength of a single angle given in the Manual appropriate for unequal legs not loaded through the shear center?

A good reference for this question is a paper by Tide, Raymond H. R. And Norbert V. Krogstad, Economical Design of Shelf Angles, Masonry: Design and Construction, Problems and Repair, ASTM STP 1180, John Melander and Lynn R. Lauersdorf, Eds., American Society of Testing and Materials, Philadelphia, PA, 1993, p. 60.

R. H. R. Tide
Wiss, Janney, Elstner Associates, Inc.
Northbrook, IL

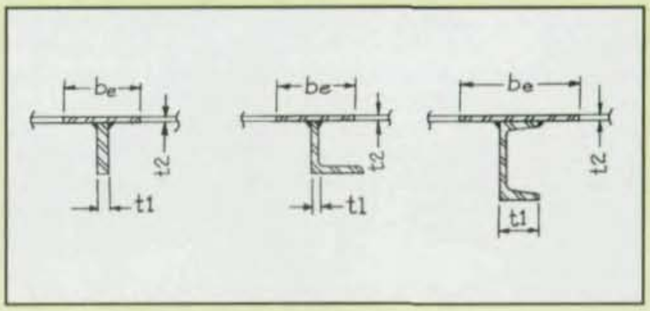
New Questions

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

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

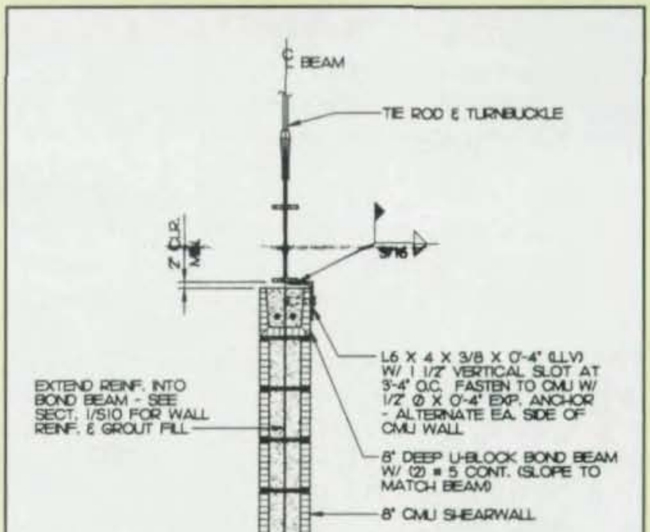
Questions and responses will be printed in future editions of Steel Interchange. Also, if you have a question or problem that readers might help solve, send these to the Steel Interchange Editor.

Given a wall of sheet metal or plate subjected to fluid pressure and stiffened by same size parallel members spaced regularly, what section (or width) of the wall shall be used that contributes to the section of a stiffener? The stiffening member may be a flat bar, an angle, a channel (see figure) or any other section.



Fran M. Lacsina
Melrose Metals
Freemont, CA

Is there a more efficient and cost-effective way to connect a masonry shear wall to structural steel framing? The most common problem with the following detail is that once the masonry is built up to the bottom flange of the beam, there is not enough room to install the grout and continuous reinforcing bars in the bond beam at the top of the wall. If the bond beam is dropped a course in elevation, the masonry to steel beam connecting angle vertical leg or bent plate vertical leg becomes excessively long.



Charles L. Bowman
Morrison and Sullivan Engineers
Raleigh, NC

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.

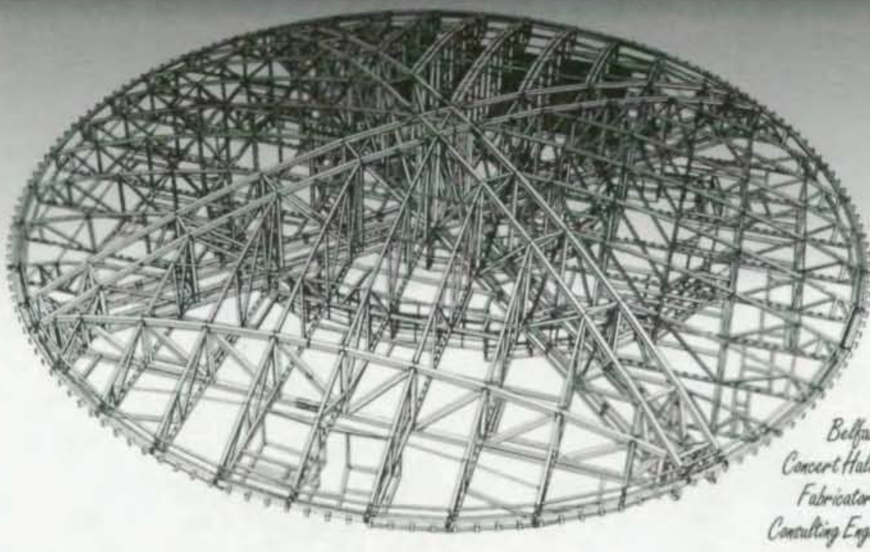
***What about a hot dip galvanized 1/4" bolt. We can do it* because we operate the only "High Temperature" kettle for centrifuge work.**

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.



Automate repetitive design tasks

1. When you type the column size

2. These values are automatically called up from the AISC Database.

This screen was created in Microsoft Excel™ to calculate base-plate thickness. Since it incorporates the AISC Database, when the column size is entered (Step 1), the column dimensions are automatically called up from the AISC Database (Step 2). These dimensions are used to complete the base-plate thickness calculation (Step 3) without having to look-up the dimensions from a reference and manually type them into the program.

	A	B	C	D	E	F	G	H
1	**BASE-PLATE THICKNESS CALCULATOR (AISC LRFD Manual, 2nd Ed.)**							
2	**Axial load**		1100	kips	**Column dimensions**			
3	**Column**		W12x170			depth, d	14.03	in.
4	**Base-plate steel grade**		A36			web thickness, tw	0.96	in.
5	**Concrete strength, f'c**		3	ksi		flange width, bf	12.57	in.
6	**Available concrete area**					flange thickness, tf	1.56	in.
7		Max. strong-axis dimension	30	in.	**Max. concrete area, A2**		900	in.^2
8		Max. weak-axis dimension	30	in.	**Base plate**			
9	**Base-plate dimensions**					Steel yield strength, Fy	36	ksi
10		Actual strong-axis dim., N	28	in.		Steel tensile strength, Fu	58	ksi
11		Actual weak-axis dim., B	26	in.		Minimum area	574	in.^2
12	**Cantilevered distances**					Optimum strong-axis dim.	25.6	in.
13		On strong-axis, m	7.34	in.		Optimum weak-axis dim.	22.4	in.
14		On weak-axis, n	7.97	in.		Actual area, A1	728	in.^2
15		Between flanges, lambda*n'	3.32	in.	**Intermediate quantities**			
16						Concrete capacity, phi Pp	1238	kips
17	**Minimum plate thickness, t**		2.43	in.		X	0.886	
18						lambda	1.00	

3. To complete the design calculations.

Save time. Save money. Efficiency, productivity, and unprecedented programming power are yours when you incorporate the ***AISC Database V2.0*** into your in-house routines. Whether you program in a language like BASIC or C, or create calculations in spreadsheets like Microsoft Excel™, Lotus 1-2-3™, or Borland Quattro Pro™, the ***AISC Database V2.0*** gives you electronic access to the dimensions and properties of the more than 1200 shapes in AISC's *Manual of Steel Construction* – W, M, S, and HP shapes, channels (C and MC), angles (L and 2L), tees (WT, ST, and MT), structural tubing (TS), and steel pipe (P). This ASCII text file lists over 25,000 dimensions and properties such as the web depth and thickness, flange width and thickness, area, moment of inertia, section modulus, radius of gyration, and torsional constant. The ***AISC Database V2.0*** is available in both U.S. units ($60) and Metric units ($60).

The ***AISC Database V2.0*** reflects the most current shape series. Additionally, new data has been added in the update from Version 1.

As with previous releases, the ***AISC Database V2.0*** comes with a sample search routine programmed in BASIC, a BASIC program to convert the ***AISC Database V2.0*** into spreadsheet format, and a BASIC program to print the database.

TO ORDER – call Carolyn Johnson, AISC Engineering Department Secretary at 312-670-5411.

All trademarks are property of the respective owners.

Setting Standards for Over 70 Years

Please circle # 1

Steel News

LETTERS TO THE EDITOR

DEAR EDITOR:

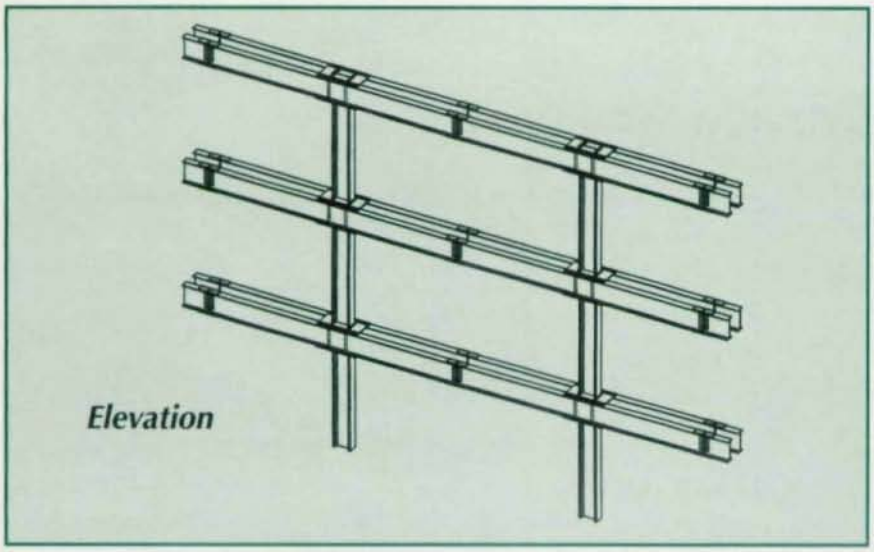
While reading the April 1995 edition I found an error in the article about galvanizing by Philip E. Rahrig with the American Galvanizers Association. On page 40 Mr. Rahrig indicates, under Number 8, that on nuts and bolts under $1^1/_2$-in. in diameter, that the bolts are threaded standard before galvanizing and that the nuts are tapped oversize after galvanizing to allow for the galvanizing thickness on the bolt threads. Mr. Rahrig then states that for fasteners over $1^1/_2$-in., the opposite practice is employed. This statement is not correct. All galvanized bolts are threaded standard and all nuts are tapped oversize after galvanizing. This practice is shown in the ASTM tables in A563 Table 5 and ASTM A307 Section 7.3. My main concern is that misinformation is not used in bolt manufacturing. If someone interprets the article as meaning the bolt thread should be undercut to accommodate the galvanizing thickness, this will result in a weaker product, as you will reduce the root diameter of the fastener, lowering its' tensile strength.

Gary Rusynyk
Portland Bolt & Manufacturing Co.

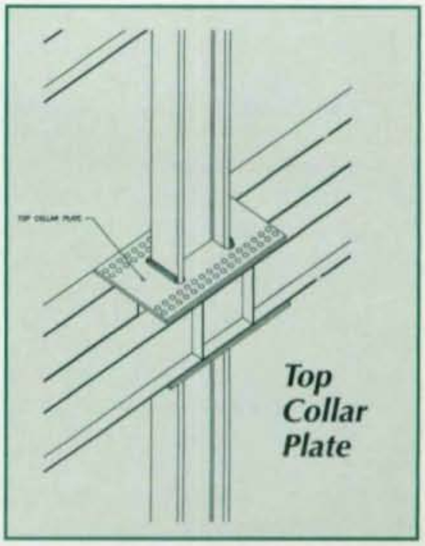
DEAR EDITOR:

On page 40, column 2, Section 8 of the April issue of Modern Steel Construction, a reference is made to ASTM A384. The reference should be to ASTM A385.

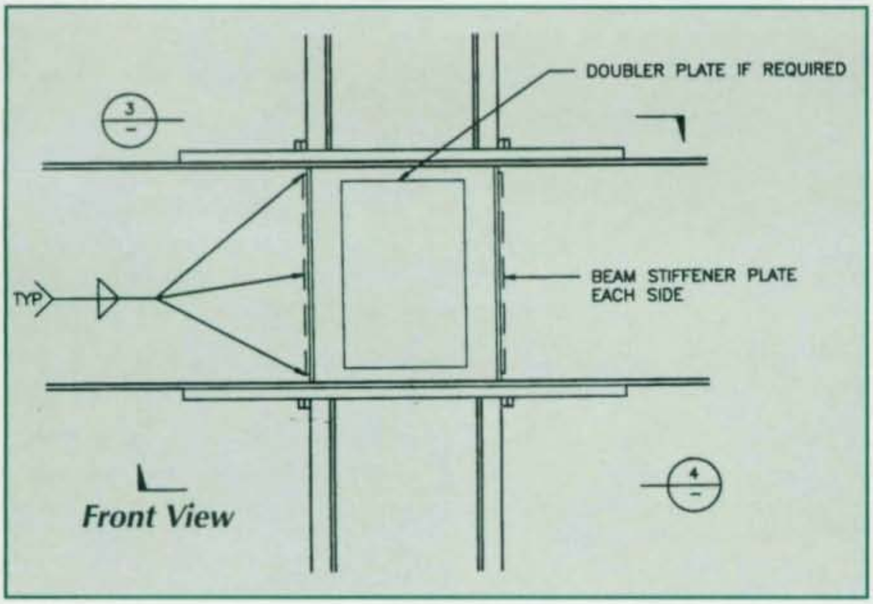
Philip G. Rahrig,
Executive / Marketing Director American Galvanizers Association



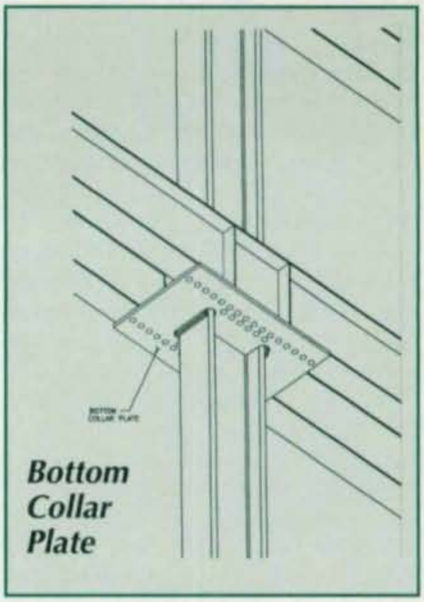
Elevation



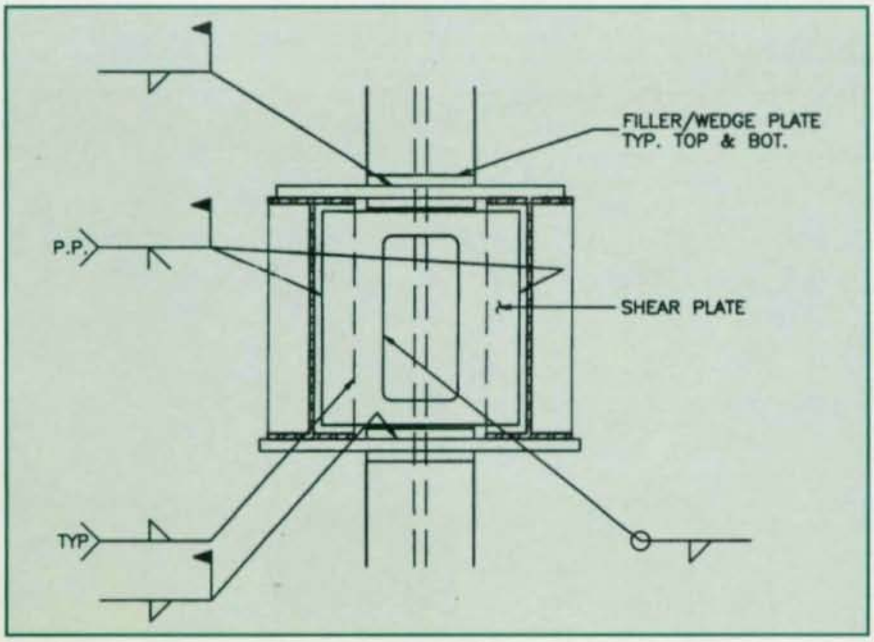
Top Collar Plate



Front View



Bottom Collar Plate



Dear Editor:

We were interested to read the article in the April 1995 edition of Modern Steel Construction entitled "Solving A Connection Dilemma" by Robert L. Boehmig. Mr. Boehmig offers an interesting concept utilizing double beams at beam column joints. There appears to be some concerns with regard to the following:

1. The adequacy of the connection between the wide flange and the channels to develop moment.

2. The adequacy of the fillet welds to develop the substantial forces that can occur.

3. Limiting the double beams

to channels.

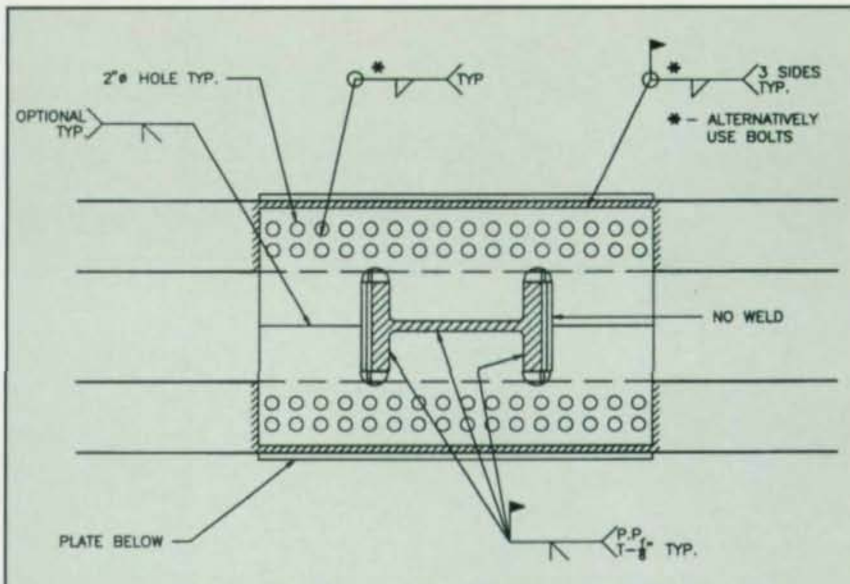
However, there appears to be some merit in Mr. Boehmig's concept of using double beams. As part of our work on seismic design, our company has been involved in looking at various options on beam column joints following the Northridge earthquake.

Prior to reading Mr. Boehmig's article, over the course of the last several months, we have developed similar ideas using double beams and no dependence on full penetration welds in tension (see drawings).

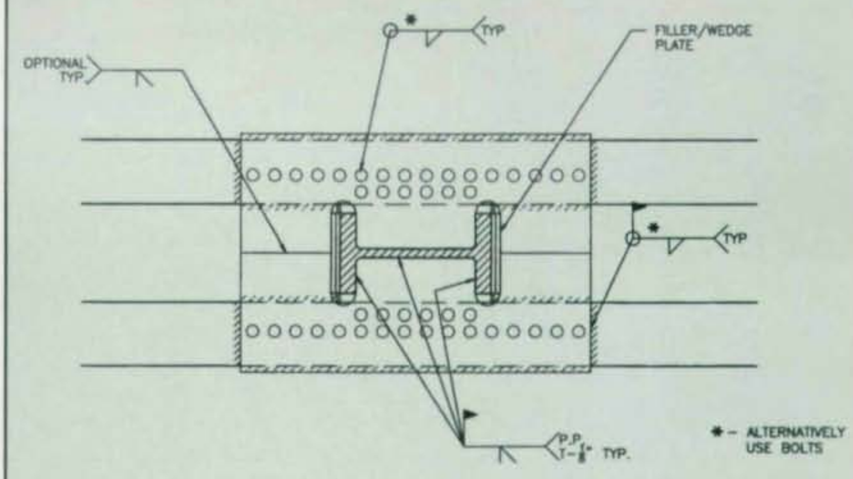
The double beams are comprised of wide flange beams and have top and bottom collar plates to transfer the forces from the beams to the columns. Transfer of substantial forces from the beam to the collar plate may be achieved by fillet welds or bolts. Transfer from the collar plate to the column is a combination of direct bearing at the column flange and welds to the column web. There is no reliance on full penetration welds in tension, avoiding the concerns for welds in tension, through flange properties, etc., which has been much debated over the course of the last year or so. Additional moment transfer is achieved from vertical shear plates welded to the column flanges and beams. The bottom collar plate can be welded in the shop and the top collar plate in the field. The beams can be field spliced at midspan where the seismic moment is small.

We recognize that the concept requires further development and testing, but offer this concept as a suggestion that may, as Mr. Boehmig's article did, also stimulate others to provide beam connection joint concepts that may one day provide the industry with improved designs.

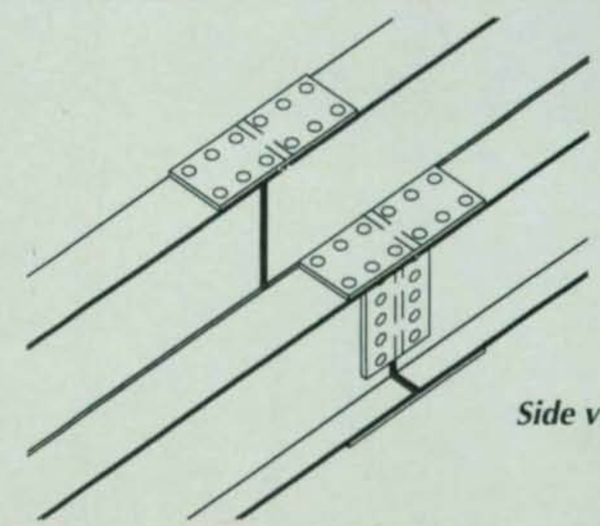
Peter J. Maranian,
Structural Engineer
Gregg. E Brandow, Ph.D.,
President
Brandow & Johnston Associates, Los Angeles



Splice at mid-span of beam—top view



View looking up at bottom flange



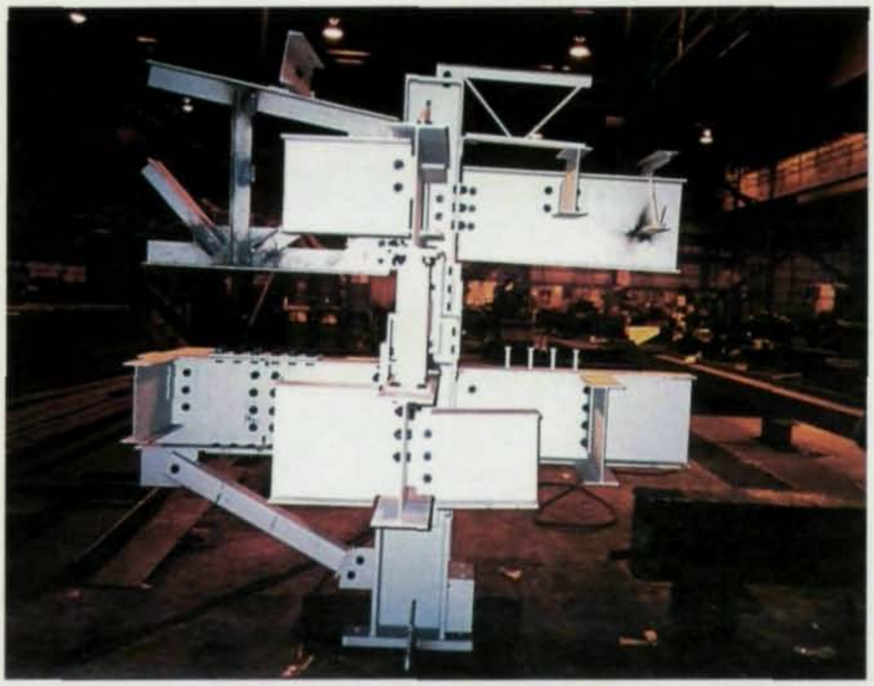
Side view

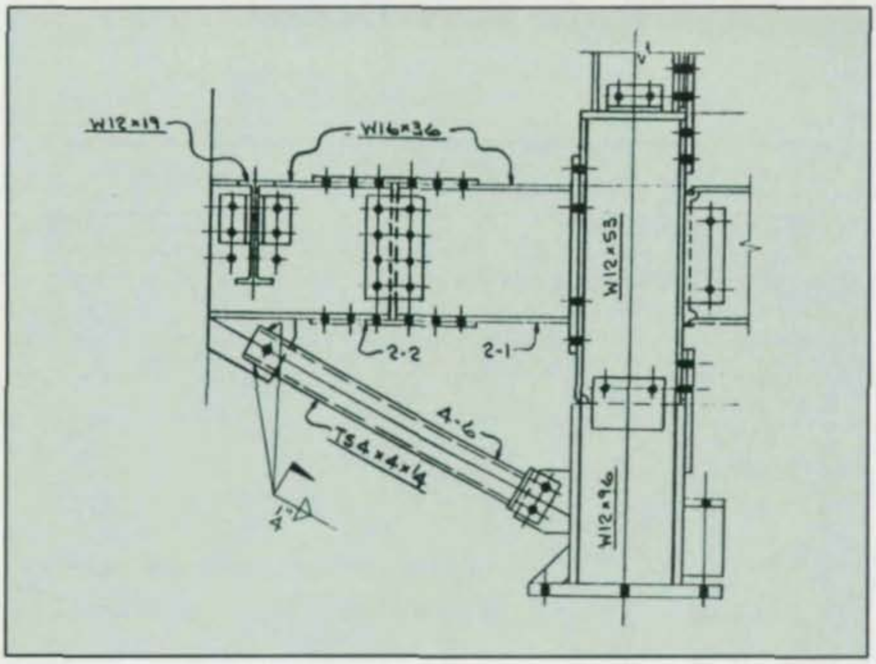
Teaching About Steel Connections

Engineering students in Wisconsin and Texas will soon have a new tool for learning about steel connections: a brightly painted, 8-ft.-high steel sculpture.

The 3,000-lb. sculpture is designed to provide students with a hands-on experience when learning about steel connections. "In a recent discussion with a structural educator, I asked him what he thought students are not exposed to, but sorely need," said Fromy Rosenberg, AISC Assistant Director of Education. "The answer came quickly: 'students need to be exposed to real situations where the three dimensional character of construction is visible to them.'"

The development of an instructional steel connections sculpture actually began nearly a decade ago in Florida. Faced with the traditional inability of students to visualize real connections, plus the fact that only a few lectures are ever devoted to connection design, Professor





Shown at top is the nearly complete, though unpainted, Wisconsin steel sculpture at AISC-member Zalk Joseph's fabrication shop.
Shown above is a drawing of the connection sculpture.

Duane Ellifritt of the University of Florida in Gainesville came up with a solution: He created a steel sculpture that would first and foremost function as a teaching aid, but which would also add to the public art on the university's campus.

"Try explaining to a student the behavior of a shop-welded, field-bolted double web angle shear connection, where the outstanding leg is made purposely long and thin so that it will flex under load and approximate a true pinned connection," Ellifrit states in explaining the value of the steel sculpture. "Text books generally show orthogonal views of such connections, but still many students have trouble in 'seeing' the real connection." In 1985, Ellifrit began investigating the possibility of using field trips to construction sites to show students actual connections. Unfortunately, many construction managers were hesitant to allow groups of students onto sites. In addition, there was the problem of only intermittent availability of construction projects. Another possibility was to build scale models, but these were rejected as too heavy to move around easily and had the further drawback of requiring a storage space when not in use.

"My eventual solution was to create a steel sculpture that would be an attractive addition to the public art already existing on campus, something that would symbolize engineering in general, and that could also function as a teaching aid," Ellifrit said. The sculpture was fabricated, erected and paid for in the fall of 1986 by AISC-member Steel Fabricators, Inc., and attracted some attention from other schools. Finally, last year, AISC created a teaching guide based on the sculpture. The guide, which is available for $5 (+$5 s&h) from AISC, shows pictures of each connection on the sculpture and offers a written description. Now, however, some local fabricators, in conjunction with nearby universities, are going a step further and completely emulating Ellifrit's original design, though on a slightly smaller scale. While the original sculpture stands more than 13-ft. high, the modified designs being built today are only 8-ft. high.

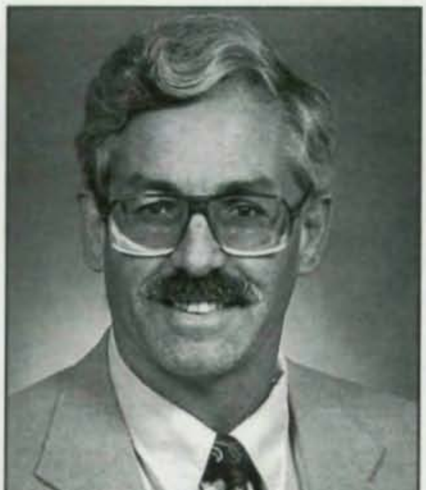
The modified plans were prepared by AISC-member Garbe Iron Works, Inc. in conjunction with several other midwest fabricators, and the first of the sculptures was recently completed by AISC-member Zalk Josephs Fabricators Inc. "The original sculpture in Florida is so large that it would be difficult for some schools to find a spot for it on campus," explained H. Louis Gurthet, president of Zalk Josephs and chairman of AISC's Committee on Education. The modified sculpture, while manageable, is still no lightweight: it weighs 3,000 lbs., is 8'x8'x8', and took between 90 and 100 hours to fabricate. As with the original sculpture in Florida, all of the material and time was donated. The sculpture is destined for the University of Wisconsin at Madison and should be installed this summer in time for the start of the fall semester.

"The sculpture will be used primarily for instructional purposes," explained the Professor Jose A. Pincheira, Jr., of the Department of Civil & Environmental Engineering at the University of Wisconsin. "I'm always struggling with teaching about the design of steel connections. We don't spend a lot of time on it, but with the steel sculpture, students can actually see the connections. Additionally, the sculpture will be on public display and should serve to attract more interest on the part of students for the design of steel connections."

Another sculpture, identical to the Wisconsin sculpture, was fabricated in Texas and will be installed on the campus of the University of Texas at San Antonio. For this sculpture, rather than one fabricator providing all of the labor, the project was divided among six fabricators, according to Ken Teel, president of AISC-member American Steel & Aluminum Co., which is coordinating the project. In addition to Teel's company, AISC members participating in the project include: Alamo Steel Co., Falcon Steel Co., and North Texas Steel Company.

To order a copy of Connecting Steel Members for $5 (+$5 s&h), call 800/644-2400 or fax 312/733-3107 and request publication G-460.

Work On Connections Garners T.R. Higgins Award



William A. Thornton

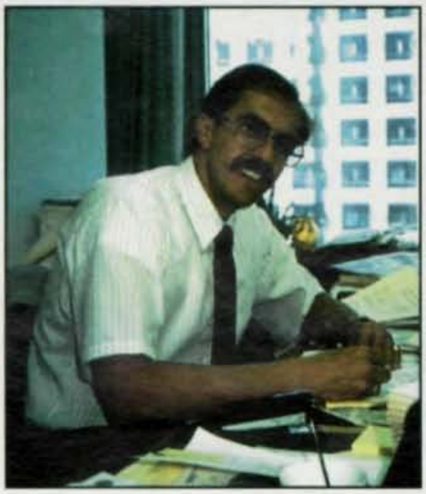
William A. Thornton, Ph.D., P.E., chief engineer of AISC-member Cives Steel Co. and president of Cives Engineering Corp., both of Roswell, GA, is the winner of the 1995 T.R. Higgins Lectureship Award. The prestigious award, given annually by AISC, recognizes an outstanding lecturer and author whose technical paper or papers are considered an outstanding contribution to the engineering literature on fabricated structural steel.

Thornton was selected for his work on connections, which is presented in his paper "Connections: Art, Science and Information in the Quest for Economics and Safety." For many *MSC* readers, the content of the paper should be somewhat familiar; much of his eminently practical information appeared in the February 1992 issue. For those unfamiliar with his work, a condensed version of this paper will run later this year and the entire paper will be printed in an upcoming issue of *Engineering Journal.*

According to Thornton, connections are an intimate part of a steel structure and their proper treatment is essential for a safe and economic structure. An intuitive knowledge of how a system will transmit loads (the art of load paths) and an understanding of structural mechanics (the science of equilibrium and of limit states) are necessary to achieve connections that are both safe and economical. Thornton's paper describes how an understanding of these principles and related information can be used to produce bracing connections, shear connections, and moment connections that satisfy both the economic and adequacy criteria.

The first public presentation of the paper was at last month's National Steel Construction Conference in San Antonio, where Thornton also received a $5,000 cash award and commemorative certificate. In addition, Thornton will make at least six presentations of the lecture in different parts of the country during 1995 and 1996. For information on these upcoming lectures, check with your local AISC Marketing regional engineer.

AISC NAMES NEW VP OF ENGINEERING



Nestor Iwankiw

NESTOR IWANKIW, FORMER DIRECTOR/RESEARCH AND CODES AND INTERIM DIRECTOR OF AISC'S ENGINEERING DEPARTMENT since September, 1994, has been promoted to Vice President, Technology and Research at AISC.

Iwankiw has 22 years of professional experience—the last 15 with AISC. Most recently, Mr. Iwankiw provided leadership and direction in post-Northridge Earthquake steel studies, development of 1992 AISC Seismic Provisions, 1993 LRFD, 2nd Edition, Specification, and joint-venture partnering on steel research with various public agencies, associations and universities.

Since joining AISC in 1980, Iwankiw has been involved in numerous research projects and development of design aids, such as eccentricity coefficients for bolted and welded groups based on ultimate strength, fillet weld strength, small column base plates, composite beams, steel fire protection, and the effects of hole-fabrication methods. He was instrumental in initiating the new AISC Design Guide

Series of publications and computer software development that supplement the AISC Manual of Steel Construction.

Iwankiw is a member of several professional societies and is a registered Professional Engineer in Illinois. He holds an MBA from the University of Chicago and a Bachelor's and Master's degree in civil engineering from the Illinois Institute of Technology. He has taught university courses in engineering mechanics and steel design as an evening instructor at IIT. Previously, he held positions with the IIT Research Institute and with Underwriters Laboratories, Inc.

Metric Guide For Steel Fabricators

A NEW PUBLICATION FROM AISC PROVIDES INFORMATION AND RESOURCES for the structural steel fabrication industry as it prepares to design using metric nomenclature—a must for those who do government contract work. *A Guide for Metric Steel Fabrication* provides basic training in metric units and offers cautionary advice until standard metric practice has been established.

Chapter 1, "Introduction to Metric," covers metric units and specific topics such as length, area and volume, weight (mass), force, stress, and floor and roof loadings. Chapter 2, "Materials," covers structural shapes, angles, plate, hollow structural sections, pipe, and high-strength bolts, nuts, and washers.

Chapter 3, "Detailing and Fabrication," deals with computer software, drafting scales, fabrication machinery, measuring tapes, welding, bolting equipment, and painting practice. Chapter 4, "Connections, gives metric design aids for bolts and welds."

Chapter 5, "Miscellaneous," summarizes preliminary metric revisions to the AISC *Code of Standard Practice* and gives a list of metric publications and resources. Finally, the Appendix tabulates conversions for length, area, mass/force, mass per unit length, and stress.

The Guide is available by calling the AISC publications line at 1-800-644-2400 and costs $15 for AISC members, $20 for non-members.

New Price For LRFD on CD

LRFD ON CD, WHICH OFFERS THE ENTIRE TWO-VOLUME *MANUAL OF STEEL CONSTRUCTION* in an easily accessible electronic form is now priced at $500 ($375 for AISC members).

The CD-ROM not only replicates the *Manual*, but improves upon it through the use of the latest "Hypertext" technology. By clicking on a word, the software automatically moves you to another reference to the same word. For example, click on "stiffeners" in the table of contents and the CD moves you immediately to page 185 in Chapter 9. From there, you can click on "local buckling" and you'll jump to page 226 in Chapter 8. That page references Table 8-49, which can then be immediately accessed. There are literally more than 1,000 cross-referenced items throughout the *Manual*.

In addition, the CD includes a 45-page introduction to LRFD electronically linked to the Specification and Commentary and nearly 100 drawings (.dxf files) taken right from the *Manual* that can be quickly copied to your AutoCAD or other CAD program.

To order a copy of this CD, call AISC's new toll-free publications number at 1-800-644-2400.

ENVIRONMENTAL COMPLIANCE WORKSHOPS

A NEW SERIES OF ENVIRONMENTAL COMPLIANCE WORKSHOPS IS BEING OFFERED THIS YEAR for steel fabricators. AISC, in conjunction with the law firm of Goldberg and Simpson in Louisville, Kentucky, will conduct six regional hands-on seminars in 1995.

The workshop is designed to be a practical, results oriented experience. Participants are encouraged to bring their own hard copy compliance materials. Of paramount concern will be compliance with the fast approaching permitting deadlines and a host of other regulations that are the focus of EPA, OSHA, and state inspections and enforcement. Workshop participants will walk away with knowledge of plan preparations, record keeping, reporting, how to handle inspections, what the potential problems are and what to do about them. Anyone responsible for completing reports or for paying the non-compliance fines would benefit from the workshops.

Among topics to be discussed are: The Clean Air Act Amendments of 1990, Storm Water Pollution Prevention

Plans, SARA Title III Emissions Reporting, Relevant State and Local Compliance Issues - geared for each local area the workshop is offered in, and Hazardous Wastes.

Each workshop runs from 8 a.m. until 4 p.m. with lunch and handouts included. The cost is $350 per company, which allows 2 attendees and $100 for each additional person.

The six regional workshops are scheduled as follows:

May 12	Salt Lake City, UT
May 26	Boxborough, MA
June 2	Valley Forge, PA
June 9	Atlanta, GA
June 16	Rosemont, IL
June 23	Columbus, OH

For more information, call 312/670-2400.

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

1995 Steel Seminar Schedule

June 20..................Charlotte
June 22..................Greenville
June 27..................Raleigh
June 28..................Norfolk
June 29..................Richmond

July 20..................Omaha
July 25..................Minneapolis
July 27..................St. Louis

August 8................Rochester, NY
August 9................Albany
August 10.............Portland, ME
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 312/670-5422 FOR INFORMATION OR A REGISTRATION FORM

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. 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 37 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 312/670-5422 or fax 312/670-5403.

First of three parts

ESSENTIALS OF LRFD

An overview of LRFD as found in Part 2 of the *Manual of Steel Construction* (1994)

NEARLY A DECADE AGO, THE AMERICAN INSTITUTE OF STEEL CONSTRUCTION, INC. (AISC) BEGAN AN INDUSTRY-WIDE TRANSITION from Allowable Stress Design (ASD) to Load and Resistance Factor Design (LRFD). While acceptance has been slow, momentum is gathering: A recent Gallup poll showed that industry acceptance of LRFD is growing and a majority of structural engineers now believe that LRFD is the steel design choice of the future. Some people, however, have interpretted the existence of two specifications as an indication of an unclear direction. Therefore, the AISC Board of Directors has adopted the following resolution:

"Based upon expert input from its Committee on Specifications, the Board of Directors of AISC affirms that the *1993 Load and Resistance Factor Design (LRFD) Specification for Structural Steel Buildings* is the preferred Specification for the fabricated structural steel industry. LRFD is a modern and technologically superior steel design specification. Its direct representation of ultimate structural behavior is especially relevant for seismic design, design of frames with partially restrained connections, and composite systems design. It offers engineers the opportunity to innovate in the analysis and design of highly reliable and competitive steel structures by encouraging the consideration of strength and serviceability criteria under appropriate combinations of gravity and lateral loads. In this way, LRFD is consistent with the prevailing trend toward limit-states design in all materials, both domestically and internationally."

This article is the first of a three-part summary of LRFD.

LRFD: AN INTRODUCTION

THE PRIMARY OBJECTIVE OF THE LRFD SPECIFICATION IS TO PROVIDE A UNIFORM RELIABILITY for steel structures under various loading conditions. This uniformity cannot be obtained with the allowable stress design (ASD) format.

The ASD method can be represented by the inequality:

$$\Sigma Q_i \leq R_n / F.S.$$

The left side is the summation of the load effects, Q_i (i.e., forces or moments). The right side is the nominal strength or resistance R_n divided by a factor of safety. When divided by the appropriate section property (e.g., area or section modulus), the two sides of the inequality become the calculated stress and allowable stress, respectively. The left side can be expanded as follows:

ΣQ_i = the maximum (absolute value) of the combinations

$D + L^1$

$(D+L^1+W) \times 0.75$*

$(D+L^1+E) \times 0.75$*

D-W

D-E

**0.75 is the reciprocal of 1.33, which represents the 1/3 increase in allowable stress permitted when wind or earthquake is taken simultaneously with like load.*

where D, L^1, W and E are, respectively, the effects of the dead, live, wind, and earthquake loads; total live load $L^1 = L + (L_r$ or S or R)

- L = Live load due to occupancy
- L_r = Roof live load
- S = Snow load
- R = Nominal load due to initial rainwater or ice exclusive of the ponding contribution

ASD, then, is characterized by the use of unfactored service loads in conjunction with a single factor of safety applied to the resistance. Because of the greater variability and, hence, unpredictability of the live load and other loads in comparison with the dead load, a uniform reliability is not possible.

LRFD, as its name implies, uses separate factors for each load and for the resistance. Considerable research and experience were needed to establish the appropriate factors. Because the different factors reflect the degree of uncertainty of different loads and combinations of loads and the accuracy of predicted strength, a more uniform reliability is possible.

The LRFD method may be summarized by the formula:

$$\Sigma \gamma_i Q_i \leq \phi R_n$$

On the left side of the inequality, the required strength is the summation of the various load effects Q_i multiplied by their respective load factors γ_i. The design strength, on the right side, is the nominal strength or resistance R_n multiplied by a resistance factor ϕ. Values of ϕ and R_n for columns, beams, etc. are provided throughout the LRFD Specification and will be covered here, as well.

According to the LRFD Specification (Section A4.1), $\Sigma \gamma_i Q_i$ = the maximum (absolute value) of the combinations

1.4D	(A4-1)
1.2D+1.6L+0.5(L_r or S or R)	(A4-2)
1.2D+1.6(L_r or S or R)+(0.5L or 0.8W)	(A4-3)
1.2D+1.3W+0.5L+0.5(L_r or S or R)	(A4-4)
1.2D +/- 1.0E+0.5L+0.2S	(A4-5)
0.9D +/- (1.3W or 1.0E)	(A4-6)

(Exception: The load factor on L in combinations A4-3, A4-4, A4-5 shall equal 1.0 for garages, areas occupied as places of public assembly, and all areas where the live load is greater than 100 psf).

The loads should be taken from the governing building code or from ASCE 7, *Minimum Design Loads in Buildings and Other Structures* (American Society of Civil Engineers, 1994). Where applicable, L should be determined from the reduced live load specified for the given member in the governing code. Earthquake loads should be from the AISC *Seismic Provisions for Structural Steel Buildings*, which appears in Part 6 of the Manual.

In the combinations the loads or load effects (i.e., forces or moments) are:

D = dead load due to the weight of the structural elements and the permanent features on the structure
L = live load due to occupancy and moveable equipment (reduced as permitted by the governing code)
L_r = roof live load
W = wind load
S = snow load
E = earthquake load
R = nominal load due to initial rainwater or ice exclusive of the ponding contribution

LRFD FUNDAMENTALS

THE FOLLOWING IS A BRIEF DISCUSSION OF THE BASIC CONCEPTS OF LRFD. A more complete treatment of the subject is available in the Commentary on the LRFD Specification (Sections A4 and A5) and in the references cited therein.

LRFD is a method for proportioning structures so that no applicable limit state is exceeded when the structure is subjected to all appropriate factored load combinations. Strength limit states are related to safety and load carrying capacity (e.g., the limit states of plastic moment and buckling). Serviceability limit states (e.g., deflections) relate to performance under normal service conditions. In general, a structural member will have several limit states. For a beam, for example, they are flexural strength, shear strength, vertical deflection, etc. Each limit state has associated with it a value of R_n, which defines the boundary of structural usefulness.

Because the AISC Specification is concerned primarily with safety, strength limit states are emphasized. The load combinations for determining the required strength were given in expressions A4-1 through A4-6. (Other load combinations, with different values of γ_i are appropriate for serviceability; see Chapter L in the LRFD Specification and Commentary.)

The AISC load factors (A4-1 through A4-6) are based on ASCE 7. They were originally developed by the A58 Load Factor Subcommittee of the American National Standards Institute, ANSI, (U.S. Department of Commerce, 1980) and are based strictly on load statistics. Being material-independent, they are applicable to all structural materials. Although others have written design codes similar in format to the LRFD Specification, the AISC was the first specification group to adopt the ANSI probability-based load factors.

The AISC load factors recognize that when several loads act in combination, only one assumes its maximum lifetime value at a time, while the others are at their "arbitrary-point-in-time" (APT) values. Each combination models the total design loading condition when a different load is at its maximum:

Load Comb.	Load at its Lifetime (50-year) Max.
A4-1	D (during const.; other loads not present)
A4-2	L
A4-3	L_r or S or R (a roof load)
A4-4	W (acting in direction of D)
A4-5	E (acting in direction of D)
A4-6	W or E (opposing D)

The other loads, which are APT loads, have mean values considerably lower than the lifetime maximums. To achieve a uniform reliability, every factored load (lifetime maximum or APT) is larger than its mean value by an amount depending on its variability.

The AISC resistance factors are based on research recommendations published by Washington University in St. Louis (ASCE Journal of Structural Division, Sept. 1978) and reviewed by the AISC Specification Advisory Committee. Test data were analyzed to determine the variability of each resistance. In general, the resistance factors are less than one ($\phi<1$). For uniform reliability, the greater the scatter in the data for a given resistance, the lower its ϕ factor.

Several representative LRFD ϕ factors for steel members (referenced to the corresponding chapters in the LRFD Specification) are:

ϕ_t = 0.90 for tensile yielding (Chapter D)
ϕ_t = 0.75 for tensile fracture (Chapter D)
ϕ_c = 0.85 for compression (Chapter E)
ϕ_b = 0.90 for flexure (Chapter F)
ϕ_v = 0.90 for shear yielding (Chapter F)

Resistance factors for other member and connection limit states are given in the LRFD Specification.

The following sections (A through I) summarize and explain the corresponding chapters of the LRFD Specification.

A. GENERAL PROVISIONS

In the LRFD Specification, Sections A4 and A5 define Load and Resistance Factor Design. The remainder of Chapter A contains general provisions which are essentially the same as in the earlier ASD editions of the Specification.

Reference is made to the *Code of Standard Practice for Steel Buildings and Bridges* (adopted in 1992 by AISC), which appears with a Commentary in Part 6 of the LRFD Manual. The Code defines the practices and commonly accepted standards in the structural steel fabricating industry. In the absence of other instructions in the contract documents, these trade practices govern the fabrication and erection of structural steel.

The types of construction recognized by the AISC Specification have not changed, except that both "simple framing" (formerly Type 2) and "semi-rigid framing" (formerly Type 3) have been combined into one category, Type PR (partially restrained). "Rigid framing" (formerly Type 1) is now Type FR (fully restrained). Type FR construction is permitted unconditionally. Type PR is allowed only upon evidence that the connections to be used are capable of furnishing, as a minimum, a predictable portion of full end restraint. Type PR construction may necessitate some inelastic, but self-limiting, deformation of a structural steel part. When specifying Type PR construction, the designer should take into account the effects of reduced connection stiffness on the stability of the structure, lateral deflections, and second order bending moments.

PR (semi-rigid) connections, once common, are again becoming popular. They offer economies in connection fabrication (compared with FR connections) and reduced member size (compared with simple framing). For information on connections, please refer to Volume II of the LRFD Manual.

The yield stresses of the grades of structural steel approved for use range from 36 ksi for the common A36 steel to 100 ksi for A514 steel. Not all rolled shapes and plate thicknesses are available for every yield stress. Availability tables for structural shapes, plates and bars are at the beginning of Part 1 of the LRFD Manual.

A36, for many years the dominant structural steel for buildings, is being replaced by the more economical 50 ksi steels. ASTM designations for structural steels with 50 ksi yield stress are: A572 for most applications, A529

Example A-1

Given: Roof beams W16x31, spaced 7'-0" center-to-center, support a superimposed dead load of 40 psf. Code specified roof are 30 psf downward (due to roof live load, snow or rain) and 20 psf upward or downward (due to wind). Determine the critical loading for LRFD.

Solution:

D	= 31 plf + 40 psf x 7.0 ft.	= 311 plf
L	= 0	
(L_r or S or R)	= 30 psf x 7.0 ft.	= 210 plf
W	= 20 psf x 7.0 ft.	= 140 plf
E	= 0	

Load Comb.	Factored Loads	
A4-1	1.4 (311 plf)	= 435 plf
A4-2	1.2(311 plf) + 0 + 0.5(210 plf)	= 478 plf
A4-3	1.2(311 plf) + 1.6 (210 plf) + 0.8(140 plf)	= 821 plf
A4-4	1.2(311 plf) + 1.3(140 plf) + 0 + 0.5(210 plf)	= 660 plf
A4-5	1.2(311 plf) + 0 + 0 + 0.2(210 plf)	= 415 plf
A4-6a	0.9(311 plf) + 1.3(140 plf)	= 462 plf
A4-6b	0.9(311 plf) - 1.3(140 plf)	= 98 plf

The critical factored load combination for design is the third, with a total factored load of 821 plf.

Example A-2

Given: The axial loads on a building column resulting from the code specified service loads have been calculated as: 100 kips from dead load, 150 kips from (reduced) floor live load, 30 kips from the roof (L_r or S or R), 60 kips due to wind, and 50 kips due to earthquake. Determine the required strength of this column.

Solution:

Load Comb.	Factored Axial Load	
A4-1	1.4(100 kips)	= 140 kips
A4-2	1.2(100 kips) + 1.6(150 kips) + 0.5(30 kips)	= 375 kips
A4-3a	1.2(100 kips) + 1.6(30 kips) + 0.5(150 kips)	= 243 kips
A4-3b	1.2(100 kips) + 1.6(30 kips) + 0.8(60 kips)	= 216 kips
A4-4	1.2(100 kips) + 1.4(60 kips) + 0.5(150 kips) + 0.5(30 kips)	= 288 kips
A4-5a	1.2(100 kips) + 1.0(50 kips) + 0.5(150 kips) + 0.2(30 kips)	= 251 kips
A4-5b	1.2(100 kips) - 1.0(50 kips) + 0.5(150 kips) + 0.2(30 kips)	= 151 kips
A4-6a	0.9(100kips) +1.3(60 kips)	= 168 kips
A4-6b	0.9(100 kips) - 1.3(60 kips)	= 12 kips
A4-6c	0.9(100 kips) + 1.0(50 kips)	= 140 kips
A4-6d	0.9(100 kips) - 1.0(50 kips)	= 40 kips

The required strength of the column is 375 kips based on the second combination of factored axial loads. As none of the results are negative, net tension need not be considered in the design of this column.

for thin-plate members only, and A242 and A588 weathering steels for atmospheric corrosion resistance. A more complete explanation is provided by Table 1-1 in Part 1 of the Manual. However, A36 is still normally specified for connection material, where no appreciable savings can be realized from higher strength steels.

Complete and accurate drawings and specifications are necessary for all stages of steel construction. The requirements for design documents are set forth in Section A7 of the LRFD Specification and Section 3 of the AISC *Code of Standard Practice*. When beam end reactions are not shown on the drawings, the structural steel detailer will refer to the appropriate tables in Part 4 of the LRFD Manual. These tables, which are for uniform loads, may significantly underestimate the effects of the concentrated loads. The recording of beam end reactions on design drawings, which is recommended in all cases, is, therefore, absolutely essential when there are concentrated loads. Beam reactions, column loads, etc., shown on design drawings should be the required strengths calculated from the factored load combinations and should be so noted.

Loads and Load Combinations

LRFD Specification Sections A4 (Loads and Load Combinations) and A5 (Design Basis) describe the basic criteria of LRFD. This information was discussed above under Introduction to LRFD.

Whether the loads themselves or the load effects are combined, the results are the same, provided the principle of superposition is valid. This is usually true because deflections are small and the stress-strain behavior is linear elastic; consequently, second order effects can usually be neglected. (The analysis of second order effects is covered in Chapter C of the LRFD Specification.) The linear elastic assumption, although not correct at the strength limit states, is valid under normal in-service loads and is permissible as a design assumption under the LRFD Specification. In fact, the Specification (in Section A.5.1) allows the designer the option of elastic or plastic analysis using the factored loads. However, to simplify this presentation, it is assumed that the more prevalent elastic analysis option has been selected.

B. Design Requirements

Gross, Net, and Effective Net Areas for Tension Members

The concept of effective net area, which in earlier editions of the Specification was applied only to bolted members, has been extended to cover members connected by welding as well. As in the past, when tensile forces are transmitted directly to all elements of the member, the net area is used to determine stresses. However, when the tensile forces are transmitted through some, but not all, of the cross-sectional elements of the member, a reduced effective net area A_e is used instead. According to Section B3 of the LRFD Specification

$$A_e = AU \qquad \text{(B3-1)}$$

where

A = area as defined below

U = reduction coefficient

= $1 - (\bar{x}/L)$ less than or equal to 0.9, or as defined in (c) or (d) (B3-2)

$\bar{x}$ = connection eccentricity. (See Commentary on the LRFD Specification, Section B3 and Figure C-B3.1)

L = length of connection in the direction of loading

a. *When the forces are transmitted only by bolts*

$A = A_n$

= net area of member, sq. in.

b. When the froces are transmitted by longitudinal welds only or in combination with transverse welds

$A = A_g$

= gross area of member, sq. in.

c. When the forces are transmitted only by transverse welds
  A = area of directly connected elements, sq. in.
  U = 1.0

d. When the forces are transmitted to a plate by longitudinal welds along both edges of the plate
  A = area of plate, sq. in.
  $l \varnothing w$
  For $l \varnothing 2w$, $U = 1.00$
  For $2w \varnothing l \varnothing 1.5w$, $U = 0.87$
  For $1.5w > l \varnothing w$, $U = 0.75$
  where
  l = weld lengths
  w = plate width (distance between welds), in.

In computing the net area for tension and shear, the width of a bolt hole is taken as $^1/_{16}$-in greater than the nominal dimension of the hole, which, for standard holes, is $^1/_{16}$-in larger than the diameter of the bolt. Chains of holes, treated as in the past, are covered in Section B2 of the LRFD Specification.

Gross, Net, and Effective Net Areas for Flexural Members

Gross areas are used for elements in compression, in beams and columns. According to Section B10 of the LRFD Specification, the properties of beams and other flexural members are based on the gross section (with no deduction for holes in the tension flange) if

$$0.75 F_u A_{fn} \varnothing 0.9 F_y A_{fg} \qquad \text{(B10-1)}$$

where
  A_{fg} = gross flange area, sq. in.
  A_{fn} = net flange area (deducting bolt holes), sq. in.
  F_y = specified yield strength, ksi
  F_u = minimum tensile strength, ksi

Otherwise, an effective tension flange area A_{fe} is used to calculate flexural properties

$$A_{fe} = \frac{5}{6}\frac{F_u}{F_y} A_{fn} \qquad \text{(B10-3)}$$

Local Buckling

Steel sections are classified as either compact, noncompact, or slender element sections:

•If the flanges are continuously connected to the web and the width-thickness ratios of all the compression elements do not exceed λ_p, then the section is compact.

•If the width-thickness ratio of at least one of its compression elements exceeds λ_p, but does not exceed λ_r, the section is noncompact.

•If the width-thickness ratio of any compression element exceeds λ_r, that element is called a slender compression element.

Columns with compact and noncompact cross sections are covered by Chapter E of the LRFD Specification. Column cross sections with slender elements require the special design procedure in Appendix B5.3 of the Specification.

Beams with compact sections are covered by Chapter F of the LRFD Specification. All other cross sections in bending must be designed in accordance with Appendices B5.3, F1 and/or G.

In general, reference to the appendices of the Specification is required for the design of members controlled by local buckling. In slender element sections, local buckling, occurring prior to initial yielding, will limit the strength of the member. Noncompact sections will yield first, but local buckling will precede the development of a fully plastic stress distribution. In actual practice, such cases are not common and can be easily avoided by designing so that:

• for beams, the width-thickness ratios of all compression elements are less than or equal to λ_p

• for columns, the width-thickness ratios of all elements less than or equal to λ_r.

Table B-1, which is an abridged version of Table B5.1 in the LRFD Specification, should be useful for this purpose. The formulas for λ_p for beam elements and λ_r for column elements are tabulated, together with the corresponding numerical values for 50 ksi steel. The definitions of "width" for use in determining the width-thickness ratios of the elements of various structural shapes are stated in Section B5 of the LRFD Specification. They are shown graphically in Figure B-1. Compact section criteria for W shapes and other I-shaped cross sections are listed in the Properties Tables in Part 1 of LRFD Manual.

Table B-1.
Limiting Width-Thickness Ratios for Compression Elements*

Beam Element	Width-Thickness Ratio	Limiting Width-Thickness Ratio, λ_p	
		General	For F_y = 50 ksi
Flanges of I shapes and channels	b/t	$65/\sqrt{F_y}$	9.2
Flanges of square and rectangular box beams	b/t	$190/\sqrt{F_y}$	26.9
Webs in flexural compression	h/t_w	$640/\sqrt{F_y}$	90.5
Webs in combined flexural and axial compression	h/t_w	$253/\sqrt{F_y}$**	35.8
Column Element	**Width-Thickness Ratio**	**Limiting Width-Thickness Ratio, λ_r**	
		General	**For F_y = 50 ksi**
Flanges of I shapes and channels and plates projecting from compression elements	b/t	$95/\sqrt{F_y}$	13.4
Webs in axial compression	h/t_w	$253/\sqrt{F_y}$	35.8

*For the complete table, see LRFD Specification, Section B5, Table B5.1.
**This is a simplified, conservative version of the corresponding entry in Table B5.1 of the LRFD Specification.

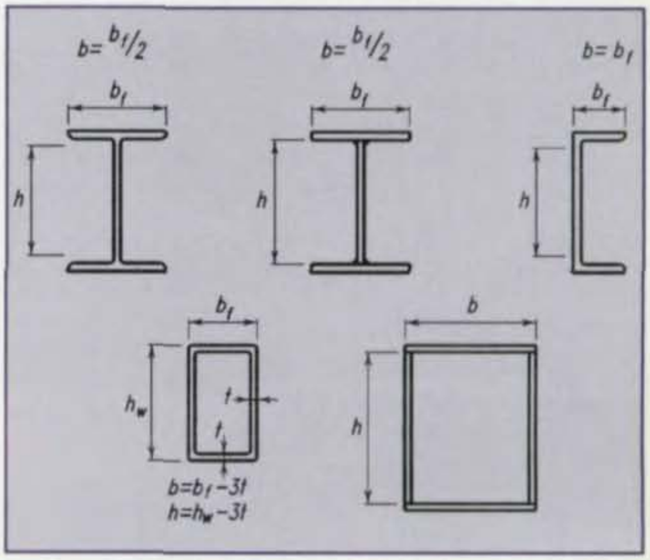


Figure B-1: Definitions of widths (b and h) for use with Table B-1

Limiting Slenderness Ratios

For members whose design is based on compressive force, the slenderness ratio Kl/r preferably should not exceed 200.

For members whose design is based on tensile force, the slenderness ratio l/r preferably should not exceed 300. The above limitation does not apply to rods in tension.

K = effective length factor, defined in Section C

l = distance between points of lateral support (l_x or l_y), in.

r = radius of gyration (r_x or r_y), in.

C. Frames and Other Structures

Second Order Effects

As stated in Section C1 of the LRFD Specification, an analysis of second order effects is required; i.e., the additional moments due to the axial loads acting on the deformed structure must be considered. In lieu of a second order analysis for M_u, the required flexural strength, the LRFD Specification (in Section C1) presents the following simplified method:

$$M_u = B_1 M_{nt} + B_2 M_{lt}$$

The components of the total factored moment, determined from a first order elastic analysis (neglecting second order effects) are divided into two groups, M_{nt} and M_{lt}. Each group is in turn multiplied by a magnification factor B_1 or B_2 and the results are added to approximate the actual second order factored moment M_u. (The method, as explained here, is valid where the moment connections are Type FR, fully restrained. The analysis for Type PR, or partially restrained, moment connections is beyond the scope of this section.)

Beam-columns are generally columns in frames, which are either braced (M_{lt} equals 0) or unbraced (M_{lt} not equal to 0). M_{nt} is the moment in the member assuming there is no lateral translation of the frame; M_{lt} is the moment due to lateral translation. M_{nt} includes the moments resulting from the gravity loads, as determined manually or by computer, using one of the customary (elastic, first order) methods. The moments from the lateral loads are classified as M_{lt}; i.e., due to lateral translation. If both the frame and its vertical loads are symmetric, M_{lt} from the vertical loads is zero. However, if either the vertical loads or the frame is asymmetric and the frame is not braced, lateral translation occurs and M_{lt} is not equal to 0. The procedure for obtaining M_{lt} in this case involves:

a. applying fictitious horizontal reactions at each floor level to prevent lateral translation, and

b. using the reverse of these reactions as the "sway forces" for determining M_{lt}.

In general, M_{lt} for an unbraced frame is the sum of the moments due to the lateral loads and these "sway forces," as illustrated in Figure C-1.

The magnification factors applied to M_{nt} and M_{lt} are, respectively, B_1 and B_2. As shown in Figure C-2, B_1 accounts for the secondary Pδ member effect in all frames (including sway-inhibited) and B_2 covers the PΔ story effect in unbraced frames. The expressions for B_1 and B_2 follow:

$$B_1 = \frac{C_m}{(1 - P_u / P_{e1})} \geq 1.0 \qquad \text{(C1-2)}$$

where:

P_u = the factored axial compressive force on the member, kips

P_{e1} = P_e as listed in Table C-1 as a function of the slenderness ratio Kl/r, with effective length factor K = 1.0 and considering l/r in the plane of bending only

l = unbraced length of the member, in.

r = radius of gyration of ts cross section, in.

C_m = a coefficient to be taken as follows:

a. For compression members not subject to transverse loading between their supports in the plane of bending,

$$C_m = 0.6 - 0.4(M_1/M_2) \qquad \text{(C1-3)}$$

where M_1/M_2 is the ratio of the smaller to larger moment at the ends of that portion of the member unbraced in the plane of bending under consideration. M_1/M_2 is positive when the member is bending in reverse curvature, negative when bending in single curvature.

b. For compression members subjected to transverse loading between their supports, the value of C_m can be determined by rational analysis, or the following values may be used:

for members with ends restrained against rotation: $C_m = 0.85$

for members with ends unrestrained against rotation: $C_m = 1.0$

Two alternative equations are given for B2 in the LRFD Specification:

$$B_2 = \frac{1}{1 - \frac{\Sigma P_u}{\Sigma H}\left(\frac{\Delta_{oh}}{L}\right)} \qquad \text{(C1-4)}$$

$$B_2 = \frac{1}{1 - \frac{\Sigma P_u}{\Sigma P_{e2}}} \qquad \text{(C1-5)}$$

where

ΣP_u = required axial strength of all columns in a story, i.e., the total factored load above that level, kips

Δ_{oh} = translational deflection of the story under consideration, in.

ΣH = sum of all story horizontal forces producing Δ_{oh}, kips

ΣP_{e2} = The summation of P_{e2} for all rigid columns in a story; P_{e2} is determined from Table C-1, considering the actual slenderness ratio Kl/r of each column in its plane of bending

K = effective length factor (see below)

Of the two expressions for B_2, the first (Equation C1-4) is better suited for design office practice. The quantity (Δ_{oh} / L) is the story drift index. For many structures, particularly tall buildings, a maximum drift index is one of the design criteria. Using this value in Equation C1-4 will facilitate the evaluation of B_2.

In general, two values of B_2 are obtained for each story of a building, one for each of the major directions. B_1 is evaluated separately for every column; two values of B_1 are needed for biaxial bending. Using Equations C1-1 through C1-5, the appropriate M_{ux} and M_{uy} are determined for each column.

Effective Length

As in previous editions of the AISC Specification, the effective length of Kl is used (instead of the actual unbraced length l) to account for the influence of end-conditions in the design of compression members. A number of acceptable methods have been utilized to evaluate K, the effective length factor. They are discussed in

Section C2 of the Commentary on the LRFD Specification. One method will be shown here.

Table C-2, which is also Table C-C2.1 in the Commentary, is taken from the Structural Stability Research Council (SSRC) *Guide to Stability Design Criteria for Metal Structures*. It relates K to the rotational and translational restraints at the ends of the column. Theoretical values for K are given, as well as the recommendations of the SSRC. The basic case is d, the classical pin-ended column, for which K = 1.0. Theoretical K values for the other cases are determined by the distances between points of inflection. The more conservative SSRC recommendations reflect the fact that perfect fixity can never be attained in actual structures.

Like its predecessors, the LRFD Specification (in Section C2) distinguishes between columns in braced and unbraced frames. In braced frames, sidesway is inhibited by attachment to diagonal bracing or shear walls. Cases a, b, and d in Table C-2 represent columns in braced frames; K is less than or equal to 1.0. The LRFD Specification requires that for compression members in braced frames, K "shall be taken as unity, unless structural analysis shows that a smaller value may be used." Common practice is to assume conservatively K = 1.0 for columns in braced frames and compression members in trusses.

The other cases in Table C-2 c, e, and f, are in unbraced frames (sidesway uninhibited); K is greater than or equal to 1.0. The SSRC recommendations given in Table C-2 are appropriate for design.

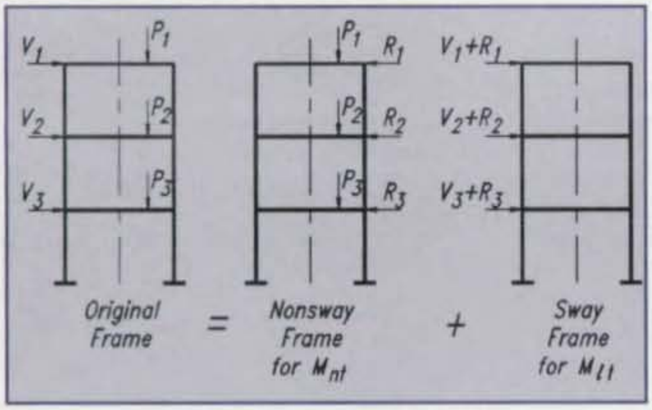


Figure C-1: Frame models for M_{nt} and M_{lt}

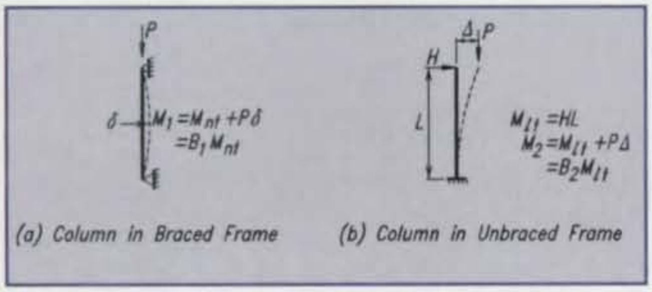


Figure C-2: Illustrations of secondary effects

"Leaning" Columns

The concept of the "leaning" column, although not related exclusively to LRFD, is new to the 1993 LRFD Specification. A leaning column is one which is pin ended and does not participate in providing lateral stability to the structure. As a result it relies on the columns in other parts of the structure for stability. In analyzing and designing unbraced frames, the effects of the leaning columns must be considered (as required by Section C2.2 of the LRFD Specification). For further information the reader is referred to:

(1) Part 3 of the Manual.
(2) the Commentary on the LRFD Specification, Section C2, and
(3) a paper on this subject (Geschwindner, 1993, AISC *Engineering Journal*, 2nd Quarter, 1995).

Part Two of "Essentials Of LRFD" will appear in the July issue of* Modern Steel Construction *and will discuss tension members, columns and beams.

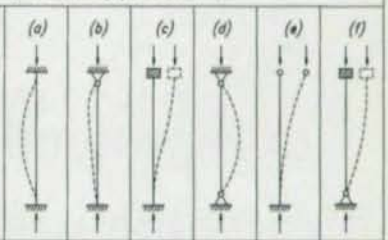
Table C-1.
Values of P_e / A_g for Use in Equation C1-2 and C1-5 for Steel of Any Yield Stress

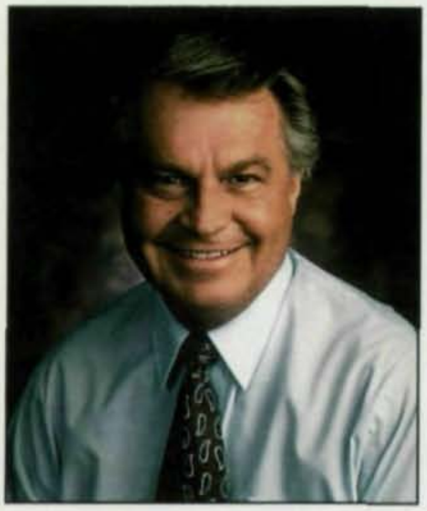
Note: Multiply tabulated values by A_g (the gross cross-sectional area of the member) to obtain P_e

Kl/r	P_e/A_g (ksi)	Kl/r	P_e/A_g (ksi)	Kl/r	P_e/A_g (ksi)	Kl/r	P_e/A_g (ksi)	Kl/r	P_e/A_g (ksi)	Kl/r	P_e/A_g (ksi)
21	649.02	51	110.04	81	43.62	111	23.23	141	14.40	171	9.79
22	591.36	52	105.85	82	42.57	112	22.82	142	14.19	172	9.67
23	541.06	53	101.89	83	41.55	113	22.42	143	14.00	173	9.56
24	496.91	54	98.15	84	40.56	114	22.02	144	13.80	174	9.45
25	457.95	55	94.62	85	39.62	115	21.64	145	13.61	175	9.35
26	423.40	56	91.27	86	38.70	116	21.27	146	13.43	176	9.24
27	392.62	57	88.08	87	37.81	117	20.91	147	13.25	177	9.14
28	365.07	58	85.08	88	36.96	118	20.56	148	13.07	178	9.03
29	340.33	59	82.22	89	36.13	119	20.21	149	12.89	179	8.93
30	318.02	60	79.51	90	35.34	120	19.88	150	12.72	180	8.83
31	297.83	61	76.92	91	34.56	121	19.55	151	12.55	181	8.74
32	279.51	62	74.46	92	33.82	122	19.23	152	12.39	182	8.64
33	262.83	63	72.11	93	33.09	123	18.92	153	12.23	183	8.55
34	247.59	64	69.88	94	32.39	124	18.61	154	12.07	184	8.45
35	233.65	65	67.74	95	31.71	125	18.32	155	11.91	185	8.36
36	220.85	66	65.71	96	31.06	126	18.03	156	11.76	186	8.27
37	209.07	67	63.76	97	30.42	127	17.75	157	11.61	187	8.18
38	198.21	68	61.90	98	29.80	128	17.47	158	11.47	188	8.10
39	188.18	69	60.12	99	29.20	129	17.20	159	11.32	189	8.01
40	178.89	70	58.41	100	28.62	130	16.94	160	11.18	190	7.93
41	170.27	71	56.78	101	28.06	131	16.68	161	11.04	191	7.85
42	162.26	72	55.21	102	27.51	132	16.43	162	10.91	192	7.76
43	154.80	73	53.71	103	26.98	133	16.18	163	10.77	193	7.68
44	147.84	74	52.57	104	26.46	134	15.94	164	10.64	194	7.60
45	141.34	75	50.88	105	25.96	135	15.70	165	10.51	195	7.53
46	135.26	76	49.55	106	25.47	136	15.47	166	10.39	196	7.45
47	129.57	77	48.27	107	25.00	137	15.25	167	10.26	197	7.38
48	124.23	78	47.04	108	24.54	138	15.03	168	10.14	198	7.30
49	119.21	79	45.86	109	24.09	139	14.81	169	10.02	199	7.23
50	114.49	80	44.72	110	23.65	140	14.60	170	9.90	200	7.16

Note: $P_e / A_g = \frac{\pi^2 E}{(Kl/r)^2}$

Table C-2.
Effective Length Factors (K) for Columns

	(a)	(b)	(c)	(d)	(e)	(f)
Buckled shape of column is shown by dashed line						
Theoretical K value	0.5	0.7	1.0	1.0	2.0	2.0
Recommended design value when ideal conditions are approximated	0.65	0.80	1.2	1.0	2.10	2.0
End condition code		Rotation fixed and translation fixed Rotation free and translation fixed Rotation fixed and translation free Rotation free and translation free				



The Image Behind The Design

By LaNae E. Dora

Richard Weingardt's firm has designed more than 3,000 projects during the past 30 years—and he's not shy about telling the world

IN AN INDUSTRY WHERE MARKETING RARELY WARRANTS EVEN A SECOND THOUGHT, Richard Weingardt, P.E., stands out. Not only is he recognized for his numerous successful projects, but he is one of the few structural engineers with a solid commitment to professionally marketing his services.

Visibility is an integral part of his strategy. In addition to being chairman of the board of Richard Weingardt Consultants Inc. (RWC) and president of the American Consulting Engineers Council, the Denver-based engineer frequently lectures and is the author of three books and more than 200 articles on creativity, leadership, business and engineering.

John Davis, P.E., president of RWC, has been with the firm for more than 17 years and is a firm supporter of Weingardt's marketing efforts. "He's very good at making the company name recognizable in the engineering field, by promoting not only our firm but the engineering discipline. Anytime your potential clients have a familiarity with the firm, that is always advantageous."

Born To Be An Engineer

Born in Sterling, CO, a town just northeast of Denver, Weingardt's initial interest in engineering stemmed from his father's work as a general contractor. "I grew up in the construction business," the now 56-year-old engineer recalls. "I liked what [my father] was doing and the engineers designed those things that he built. It just seemed natural." That desire, along with a strength in math and science, led him to the University of Colorado where he earned a bachelor's degree in civil engineering and a master's degree in structural engineering.

His first job was as a structural engineer with the U.S. Bureau of Reclamation in 1960. At the time, he says, the Bureau was actively designing dams, power plant, and other large structures—very exciting work. Weingardt spent four years there and then worked as a project engineer with Ketchum Konkel Ryan & Fleming, a Denver consulting engineering firm.

However, Weingardt always wanted to go into business for himself, as his father had. At one time he considered going working for his father, but the appeal of running an engineering company was greater. "I decided I wanted to be a consulting engineer because we design a lot of projects, whereas a contractor is more restricted to a few projects because of the capital involved."

And in the last three decades, there have been many projects for RWC—more than 3,000 in 42 states and 12 foreign countries—and the company has won more than 50 design awards. While not the largest engineering firm, RWC is still much larger than most consulting engineering practices. The company has a staff of 34, including 20 structural engineers. In contrast, the average structural office has only five or six engineers, according to Weingardt.

The size of RWC allows the company to work on larger and a greater number of projects. "We like to assign principals to a project, and that allows us to give the client some really good service."

Jim Bradburn, of Fentriss, Bradburn and Associates P.C., an architectural firm that has worked with RWC on several projects, appreciates the benefits of personal attention from the firm's principals. "They're a firm that provides exceptionally good structural engineering design, as evidenced by the awards they've won, and you get personal treatment. Those things are important to me."

A recent project that aptly illustrates all of those achievements is the Jefferson County Government Center in Colorado. "I think what sets us apart in any design is that we're willing to go to the leading edge. If something's been done before that's fine, but we're also willing to try something new," Weingardt explains. On the Government Center project, that extra innovative touch came in the design of a tension ring for the building's dome. "We came up with the concept of a hollow truss that picks up four points at the outer edge of the truss at the top and is supported by four points at the outer edge of the truss on the bottom."

When Weingardt first entered the profession, that type of design would have been next to impossible to visualize. However, by using the latest generation of computer software, such as LARSA 3D Structural Analysis Software for finite element analysis and MICAS Structural Utilities Package, RWC can expand its design envelope to encompass a wide range of three-dimensional space frames. "We don't just stick with the linear

Richard Weingardt Consultants recently won an ACEC Excellence Award for the renovation and expansion of the Cowboy Hall of Fame in Oklahoma City. To house the famous "End of Trail Sculpture" by James Earle Fraser—one of the most copied images in American western art—the existing concrete roof over an entire bay was eliminated. The three-dimensional folded concrete roof was completely severed in the middle and provented from collapsing through a unique combination of two hidden interior columns, a free-standing exterior space-frame structure and the attachment of a continuous steel tension plate to the severed edges of the slab. The combination resists all vertical and horizontal forces and prevented any damaging movemnts and deflections both during construction and in the future.

Pictured above is a view of the exposed roof in Concourse "C" at the new Denver International Airport. Shown at right is a view of the typical roof framing in Concourse "B", including the aptly named "Hero" trusses.

anymore," Weingardt says.

And it's exactly this technological originality that Weingardt's clients have come to expect from him.

"The thing I really like about him is that he's a creative thinker," says Jerry Seracuse, FAIA, a Denver architect who has worked with RWC on numerous projects for more than 25 years. "What you look for in Rich is the unique or unusual solution to a problem. He's not afraid to do them."

A good example is the space frame roof for the Highplains Easter Seal Clinic in Sterling, CO, which utilized a series of cantilevers extending from only four columns. Accomplishing the design required welding together open-web steel joists with cords perpendicular to the trusses on the top and bottom. "We were willing to try it because it was economical and could be done with whatever labor was available in that marketplace," Weingardt explained.

Another example is the Gerald Ford Amphitheatre in Vail, CO. For that project, RWC designed a "floating roof" structure. The structure features a roof with large cantilevers and utilizes free-standing roof and wall column sections that are designed to resist heavy wind and snow loads. Crucial to the design was a foundation system utilizing a unique soil tension anchor system. The design was impressive enough for RWC to be awarded an Engineering Excellence Award from the American Consulting Engineers Council of Colorado (CECC).

One of the most recent award-winning project for Weingardt was the design of the roof structures for the three concourses at the new Denver International Airport. RWC used structural steel space frames and long-span "Hero" trusses. "We had a lot of very large trusses that had to pick up transfer beams and columns," says Weingardt. The large trusses, which each support 250 tons of load, required special pin and knuckle joints to connect the trusses with cantilevered columns and to dissipate torque from differential loading conditions. They were christened "Hero" trusses because "they do everything you can ask of a structure." That creation also was recognized by the CECC.

The old Denver airport also has fond memories for Weingardt. RWC expanded a concourse at the old airport by employing an innovative structural steel system that enabled the concourse to remain open during construction. The roof of the concourse was raised and its size doubled. Working methodically, one new section was opened after which an old section was sealed off and renovated. "That was a pretty special project. The passengers never even knew what was happening."

While all of these project were exciting and required "leading edge" design work, they also rank among the seasoned engineer's favorite projects because all were economically successful and resulted in happy clients.

MARKETER EXEMPLAR

Weingardt's extensive marketing plan is key to his company's success. "Marketing to me is getting the people you want to work with to know you. The basic core philosophy we have is: People do business with people they know."

To accomplish this, RWC does everything from sending out newsletters and brochures to setting up meetings with potential clients. Because some clients are "wary of getting to know someone new," these presentations

The Idaho Light & Power office complex in Boise features column-free floors. Instead, the columns are located external to the building shell. Structural steel X-bracing within the core walls resists lateral seismic and wind loads. Also, X-bracing was used around the elevator shafts instead of concrete shear walls.

are planned around the question: "What can RWC offer this client that another company hasn't already approached them with?"

Many of the meetings result from "cold calls" that are made by the principals themselves. "We think principals ought to be talking to the people who will be hiring us. We tell them: 'We'd like to have a five- or 10-minute meeting just to introduce ourselves, get to know you face-to-face, and talk to you about a certain technique we've developed that you might want to use on one of your upcoming projects," Weingardt says.

This method, he estimates, brings in about 20 percent of the work RWC does; the other 80 percent is acquired though repeat clients. About eight percent of the firm's gross billings is spent on manhours and materials for marketing. By comparison, a 1994 survey by the Society for Marketing Professional ple, he says, attribute the total design of buildings to architects and don't even give a second thought to the engineers on a project. One of his objectives as ACEC president, he says, is to build public awareness of the consulting engineering field. He thinks consultants should also become more visible in their communities.

To facilitate this, Weingardt wants to establish four task forces to implement what he calls the four I's: Investment, Involvement, Impact and Imagination. The first group will focus on what investments ACEC members need to make to improve the quality of their work. The second unit will encourage engineers to become more involved in ACEC activities and to recruit new members. The third group will focus on how consulting engineers deal with the media and the general community. The fourth group will consist of engineering and industry leaders who can inform members about upcoming trends and help them imagine how the future will affect their field.

Weingardt believes that in the future, more engineers will head up design teams because of the partnerships needed to raise capital for projects. "I think we're going to see a lot of private/public ventures to get projects underway. I can see structural engineers as leaders in those arenas. I see engineers getting more and more into the management of design teams than they have in the past."

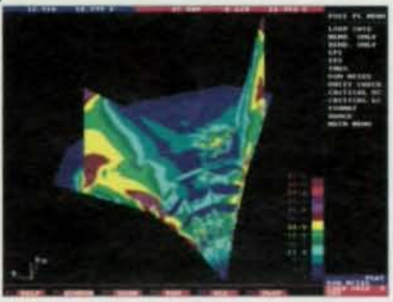
Expanding Markets

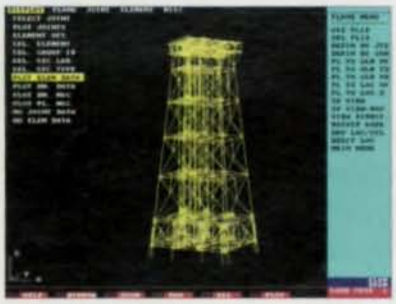
As for the future, Weingardt envisions that RWC will become more involved in the global marketplace. After his one-year term as ACEC president, he plans to come back to his firm and participate in this international expansion. To accomplish this, he plans to build upon the partnerships RWC has already made with private companies that do work overseas. RWC has already done work in Russia, South Korea, Taiwan, Saudi Arabia, and several other countries.

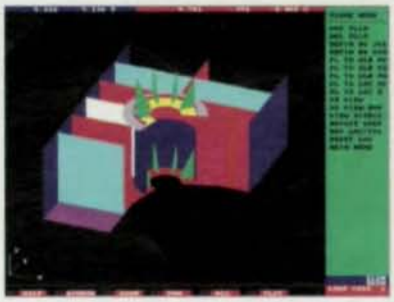
As could be expected, Weingardt also has a vision for the entire engineering community.

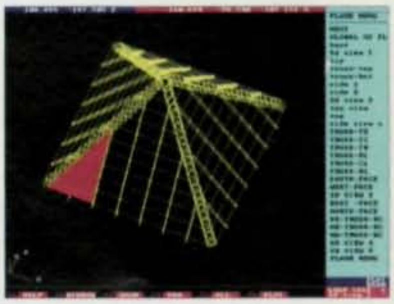
"I think consulting engineers many times react to the world around them—the new legislation, the new regulators, the new marketplace. I'd like us to better prepare for the future so that we're in a proactive stance. I'd like to get us better aware of how we can set or predict the trends—where our markets will be and what our firms will look like in the year 2000."

LaNae E. Dora is a freelance writer based in Chicago.









Services states that the average firm spends only 2.5% of its revenue on marketing.

Weingardt's marketing acumen is deeply admired and appreciated by the people who work with him. "For a company as small as his to get as much market and stay in business as long as he has is a credit to his ability to market it," says Jim Bradburn.

Jerry Seracuse agrees. "If I was as a good a marketer as Rich Weingardt, I'd be the happiest guy in the world."

Professional Involvement

Weingardt's involvement in engineering extends far beyond his own company. His public service endeavors include working on the Engineering Development Council at the University of Colorado at Boulder, the Engineering Advisory Council at the University of Colorado at Denver and the Engineering Advisory Board at the University of Denver. He is a member of such organizations as the American Society of Civil Engineers, the National Society of Professional Engineers, Professional Engineers of Colorado and the Structural Engineers Association of Colorado.

According to colleagues, leadership is nothing new to Weingardt. "He has always looked for roles of leadership that enable him to promote the profession and to attend to the issues that affect the engineering field," Davis said.

Currently, Weingardt's primary commitment is serving as president of the American Consulting Engineers Council (ACEC), a term which began just last month. As president-elect during the past year, he spent much time in Washington, DC, doing what he terms "a full-time job" meeting with officials on

The Gerald Ford Amphitheatre in Vail, CO, features a roof with large cantilevers and utilizes free-standing roof and wall column sections that are designed to resist heavy wind and snow loads.

Please circle # 71

The roof system for the High Plains Rehabilitation Center uses standard long-span steel joists tilted at an angle and then welded together using standard steel channel sections running perpendicular to the joists, creating the top and bottom chords of the space frame. Once all the channels were welded in place to the joists, the entire space frame was raised off its temporary support on the exterior masonry curtain walls. Ultimately, the entire space frame was supported by only four interior columns near each corner of the building.

issues that affect consulting engineers—a schedule that he doesn't expect to slacken this year. He also travels to ACEC chapters in all 50 states to coordinate with them and resolve any problems.

Les MacFarlane, the previous ACEC president, is confident that Weingardt will have an influence on his profession. "The process of becoming president of ACEC is a pretty rigorous one. It's not a popularity contest. To make it is quite an honor." He also has the distinction of the being the first practicing structural engineer to hold the position.

Weingardt want to increase the visibility of engineers and their projects among the general public. "Most of the infrastructure designed in the United States, if it is done by the private sector, is done by consulting engineers. Yet very few lay people know we do this." Many peo-

AN ALTERNATIVE TO JOIST GIRDERS

Talk of slow deliveries has prompted consideration of using rolled shapes in place of roof joist girders

WITH ONLY TWO MAJOR SUPPLIERS OF JOISTS, some fabricators and contractors have been increasingly vocal in complaining about what they see as slow product delivery. As a result, AISC is investigating an alternative framing method using rolled beams to replace "K" series joist girders. The system, developed by Downers Grove, IL-based consultant Mark Zahn, S.E., is arguably the most efficient method of framing with cantilevered and suspended roof girders.

The advantages of the system include:

- more suppliers than with joist girders, so there is greater flexibility in meeting a schedule;
- economy of scale when more work is performed by the steel fabricator;
- no stiffeners required on simple spans;
- a smaller beam depth than with joist girders, so the overall dimensions of a building may be reduced;
- greater versatility in carrying concentrated loads; and
- reinforcement can be easily accomplished if future modifications are made.

The major disadvantage of the system is that it is expected to have a slight cost premium over standard joist girders.

The tables included with this article offer roof framing solutions, based on a distinct set of loading, serviceability, bay size and joist span parameters. The information should be considered as an example only, and it is up to an individual designer to extrapolate the information for use on a specific project. All information should be verified for suitability and applicability on any specific project by a licensed professional structural engineer.

DESIGN PARAMETERS AND LIMITATIONS

Many specific parameters and limitations go into the design of any structural member. Imposed loadings caused by earthquake, wind, snow, rain, construction methods, etc., vary across the country. Live loads are specified in the applicable building codes. Dead loads are much more variable and require special attention in their computation. Specific requirements for serviceability, strength, lateral stability of individual elements, and the lateral resistance of the building all contribute to the design of a safe and efficient building. The information presented is intended for use in roof framing conditions without regard to earthquake loading or contributing to lateral resistance of the building.

Bay sizes presented are 30'x30', 30'x40', 40'x30', 40'x40', 40'x50', 50'x40', and 50'x50'. Five typical conditions for live and dead loads are each tabulated. Live loads address both snow and no snow regions. Dead loads address both built-up and ballasted roof systems. Connection design tables also are included. The cantilever and suspended roof girder system design tables are based on the following parameters:

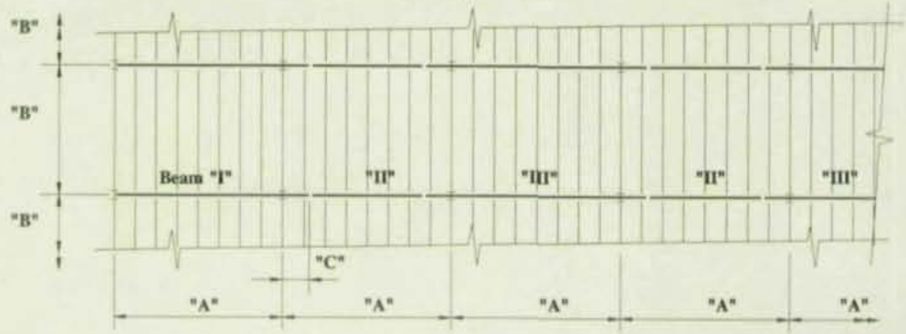
- Load and Resistance Factor Design Specification, December 1, 1994
- Roof loading is uniform on all spans
- Cantilever length is selected to provide approximately equal positive and negative moments for a uniformly loaded system
- Column spacing is uniform in each direction
- A "tie joist" is mandatory at each column line; joist and bottom chord extension are to have sufficient strength and rigidity to provide lateral torsional restraint of the girder
- Joists are uniformly spaced between columns
- Girder webs have been checked for stiffener requirements and noted only if required
- Top of columns are laterally supported by the tie joists/girder
- Columns may be wide flange, pipe, or tube having a rigid cap plate 12" (min.) in width longitudinal to the girder
- Column design, roof deck selection is not part of this presentation
- Total load deflection is limited to 1/240 of the girder span
- Roof deck/joist system provides lateral support of the girder top flange

Graphically, framing plans indicate joists that are "in-line" across the girders. Not all tabulated member flange widths will allow this condition due to joist bearing criteria. Actual member sizes may be selected with a wider flange or the joists may be staggered for full joist bearing and member economy. Final member selection is the responsibility of the engineer-of-record.

Roof joist selections are included in the design tables to complete the roof framing system. Joist girder design information is tabulated for direct comparison to the alternate cantilever and suspended girder system.

LRFD Cantilever & Suspended Roof Girder System

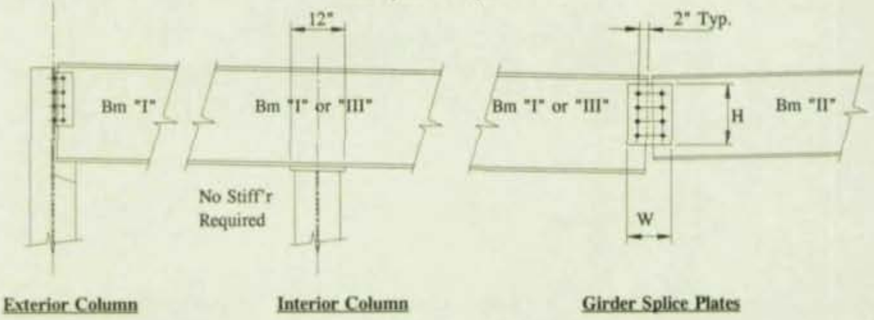
Live Load = 12 psf
Dead Load = 18 psf
Wind Uplift = 14 psf



Girder Span "A"	Joist Span "B"	Splice Dim. "C"	Recommended Wide Flange Member Design (50 ksi) Beam "I"	Beam "II"	Beam "III"	Joist Selection Designation	Spacing	Joist Girder Required Simple Span Condition Designation	Lbs./Ft.
30'	30'	6'-0"	W16x26	W12x19	W14x22	16K2	5'-0"	20G6N5K	24 ±
30'	40'	6'-0"	W16x31	W12x19	W16x26	20K4	5'-0"	20G6N6.6K	33 ±
40'	30'	6'-0"	W21x44	W16x31	W16x31	16K2	5'-0"	24G8N5K	35 ±
40'	40'	6'-0"	W21x50	W18x40	W18x40	20K4	5'-0"	28G8N6.6K	42 ±
40'	50'	6'-0"	W24x62	W21x44	W21x50	26K6	5'-0"	32G8N8K	43 ±
50'	40'	6'-0"	W24x68	W21x62	W21X62	20K4	5'-0"	32G10N6.8K	68 ±
50'	50'	6'-0"	W24x84	W21x62	W24x62	26K6	5'-0"	36G10N8K	76 ±

LRFD Cantilever & Suspended Roof Girder System

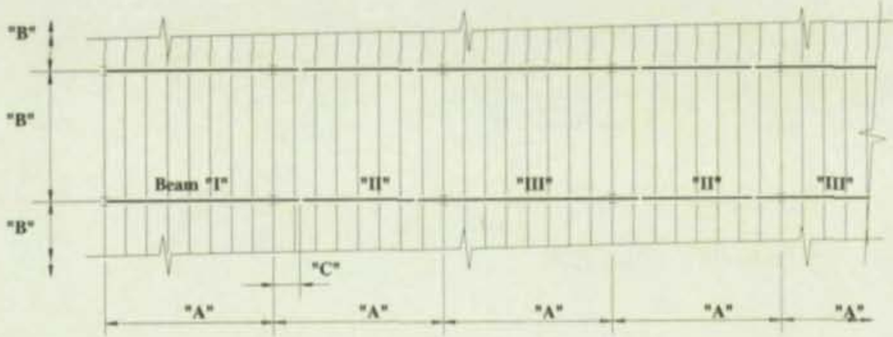
Live Load = 12 psf
Dead Load = 18 psf
Wind Uplift = 14 psf



Girder Span "A"	Joist Span "B"	Splice Dim. "C"	Splice Plates - A36 Mat'l - (1 Near Side & 1 Far Side) 3/4"ϕ A325N Bolts — Plate Size t x H x W	Bolts #/Rows @ 3" o.c.	Connection at Exterior Column A36 - Dbl. Angle / 3/4"ϕ A235N Bolts — Double Angles Thickness x Height	Bolts #/Rows @ 3" o.c.
30'	30'	6'-0"	5/16" x 7" x 0'-9"	2	1/4" x 0'-8 1/2"	3
30'	40'	6'-0"	5/16" x 7" x 0'-9"	2	1/4" x 0'-8 1/2"	3
40'	30'	6'-0"	5/16" x 10" x 0'-9"	3	1/4" x 0'-11 1/2"	4
40'	40'	6'-0"	5/16" x 10" x 0'-9"	3	1/4" x 0'-11 1/2"	4
40'	50'	6'-0"	5/16" x 10" x 0'-9"	3	1/4" x 0'-11 1/2"	4
50'	40'	6'-0"	5/16" x 10" x 0'-9"	3	1/4" x 0'-11 1/2"	4
50'	50'	6'-0"	5/16" x 10" x 0'-9"	3	1/4" x 0'-11 1/2"	4

LRFD Cantilever & Suspended Roof Girder System

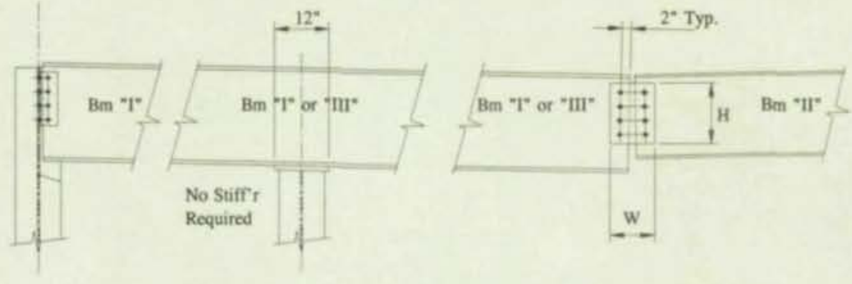
Live Load = 12 psf
Dead Load = 35 psf



Girder Span	Joist Span	Splice Dim.	Recommended Wide Flange Member Design (50 ksi)			Joist Selection		Joist Girder Required Simple Span Condition	
"A"	"B"	"C"	Beam "I"	Beam "II"	Beam "III"	Designation	Spacing	Designation	Lbs./Ft.
30'	30'	6'-0"	W16x31	W14x22	W16x26	18K4	5'-0"	28G6N7.1K	26 ±
30'	40'	6'-0"	W18x35	W16x26	W18x35	24K7	5'-0"	28G6N9.5K	35 ±
40'	30'	6'-0"	W24x55	W18x35	W18x40	18K4	5'-0"	32G8N7.1K	37 ±
40'	40'	6'-0"	W24x62	W18x40	W21x50	24K7	5'-0"	36G8N9.5K	48 ±
40'	50'	6'-0"	W24x68	W21x50	W24x55	26K10	5'-0"	40G8N12K	52 ±
50'	40'	6'-0"	W27x84	W24x68	W24x68	24K7	5'-0"	44G10N9.5K	65 ±
50'	50'	6'-0"	W30x90	W24x76	W24x84	26K10	5'-0"	52G10N12K	68 ±

LRFD Cantilever & Suspended Roof Girder System

Live Load = 12 psf
Dead Load = 35 psf

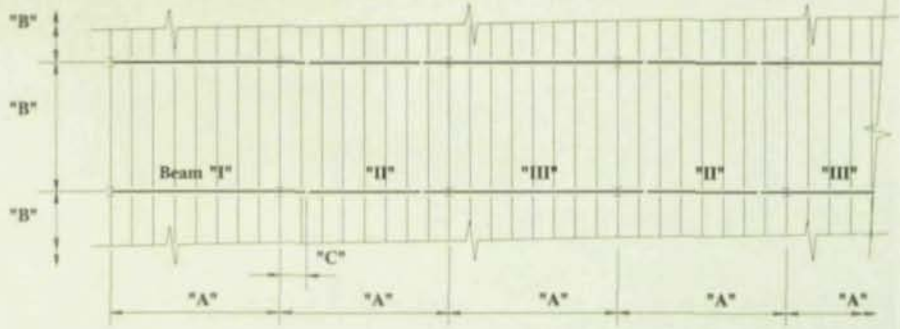


Exterior Column **Interior Column** **Girder Splice Plates**

Girder Span	Joist Span	Splice Dim.	Splice Plates - A36 Mat'l - (1 Near Side & 1 Far Side) 3/4"ϕ A325N Bolts		Connection at Exterior Column A36 - Dbl. Angle / 3/4"ϕ A235N Bolts	
"A"	"B"	"C"	Plate Size t x H x W	Bolts #/Rows @ 3" o.c.	Double Angles Thickness x Height	Bolts #/Rows @ 3" o.c.
30'	30'	6'-0"	5/16" x 7" x 0'-9"	2	1/4" x 0'-8 1/2"	3
30'	40'	6'-0"	5/16" x 10" x 0'-9"	3	1/4" x 0'-11 1/2"	4
40'	30'	6'-0"	5/16" x 10" x 0'-9"	3	1/4" x 0'-11 1/2"	4
40'	40'	6'-0"	5/16" x 10" x 0'-9"	3	1/4" x 0'-11 1/2"	4
40'	50'	6'-0"	5/16" x 10" x 0'-9"	3	1/4" x 0'-11 1/2"	4
50'	40'	6'-0"	5/16" x 10" x 0'-9"	3	1/4" x 0'-11 1/2"	4
50'	50'	6'-0"	5/16" x 13" x 0'-9"	4	1/4" x 0'-11 1/2"	4

LRFD Cantilever & Suspended Roof Girder System

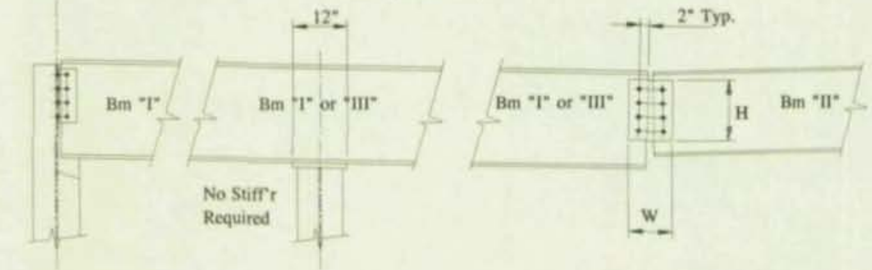
Live Load = 20 psf
Dead Load = 20 psf
Wind Uplift = 14 psf



Girder Span "A"	Joist Span "B"	Splice Dim. "C"	Recommended Wide Flange Member Design (50 ksi)			Joist Selection		Joist Girder Required Simple Span Condition	
			Beam "I"	Beam "II"	Beam "III"	Designation	Spacing	Designation	Lbs./Ft.
30'	30'	6'-0"	W16x31	W14x22	W16x26	18K3	5'-0"	20G6N6.1K	29 ±
30'	40'	6'-0"	W16x36	W14x22	W16x31	20K7	5'-0"	24G6N8.2K	33 ±
40'	30'	6'-0"	W21x50	W16x31	W18x35	18K3	5'-0"	28G8N6.1K	37 ±
40'	40'	6'-0"	W24x55	W18x40	W21x44	20K7	5'-0"	32G8N8.2K	45 ±
40'	50'	6'-0"	W24x62	W21x50	W21x50	26K9	5'-0"	36G8N10.2K	50 ±
50'	40'	6'-0"	W27x84	W24x55	W24X68	20K7	5'-0"	40G10N8.2K	58 ±
50'	50'	6'-0"	W27x94	W24x62	W24x76	26K9	5'-0"	44G10N10.2K	67 ±

LRFD Cantilever & Suspended Roof Girder System

Live Load = 20 psf
Dead Load = 20 psf
Wind Uplift = 14 psf

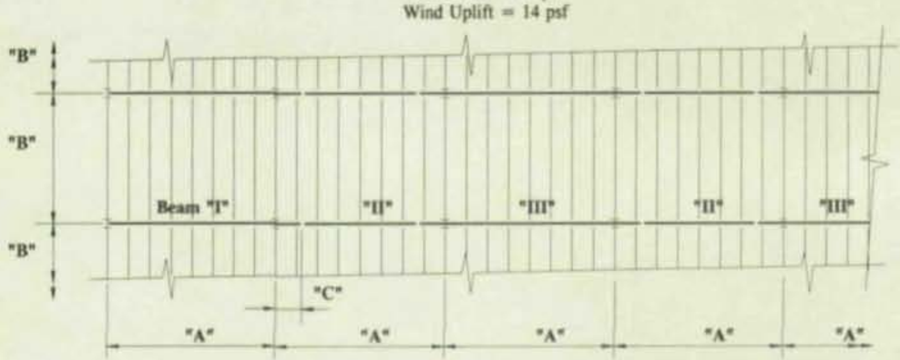


Exterior Column **Interior Column** **Girder Splice Plates**

Girder Span "A"	Joist Span "B"	Splice Dim. "C"	Splice Plates - A36 Mat'l - (1 Near Side & 1 Far Side) 3/4"ϕ A325N Bolts		Connection at Exterior Column A36 - Dbl. Angle / 3/4"ϕ A235N Bolts	
			Plate Size t x H x W	Bolts #/Rows @ 3" o.c.	Double Angles Thickness x Height	Bolts #/Rows @ 3" o.c.
30'	30'	6'-0"	5/16" x 7" x 0'-9"	2	1/4" x 0'-8 1/2"	3
30'	40'	6'-0"	5/16" x 7" x 0'-9"	2	1/4" x 0'-8 1/2"	3
40'	30'	6'-0"	5/16" x 10" x 0'-9"	3	1/4" x 0'-11 1/2"	4
40'	40'	6'-0"	5/16" x 10" x 0'-9"	3	1/4" x 0'-11 1/2"	4
40'	50'	6'-0"	5/16" x 10" x 0'-9"	3	1/4" x 0'-11 1/2"	4
50'	40'	6'-0"	5/16" x 13" x 0'-9"	4	1/4" x 0'-11 1/2"	4
50'	50'	6'-0"	5/16" x 13" x 0'-9"	4	1/4" x 0'-11 1/2"	4

LRFD Cantilever & Suspended Roof Girder System

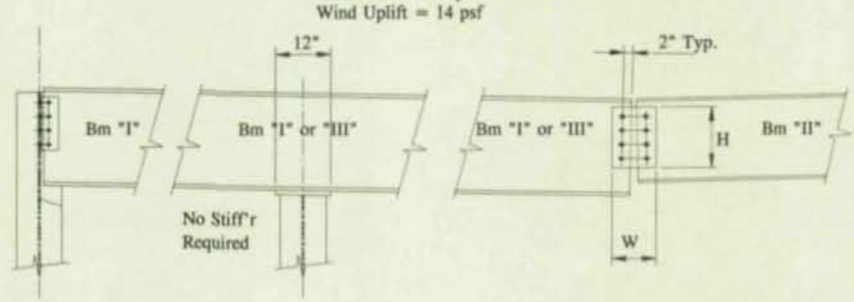
Live Load = 30 psf
Dead Load = 18 psf
Wind Uplift = 14 psf



Girder Span	Joist Span	Splice Dim.	Recommended Wide Flange Member Design (50 ksi)			Joist Selection		Joist Girder Required Simple Span Condition	
"A"	"B"	"C"	Beam "I"	Beam "II"	Beam "III"	Designation	Spacing	Designation	Lbs./Ft.
30'	30'	6'-0"	W16x31	W14x22	W16x26	20K4	5'-0"	24G6N7.7K	32 ±
30'	40'	6'-0"	W18x35	W16x26	W18x35	24K7	5'-0"	24G6N10K	39 ±
40'	30'	6'-0"	W24x55	W18x35	W18x40	20K4	5'-0"	28G8N8K	47 ±
40'	40'	6'-0"	W24x62	W18x40	W21x50	24K7	5'-0"	32G8N10K	51 ±
40'	50'	6'-0"	W24x68	W21x50	W24x55	26K10	5'-0"	40G8N13K	56 ±
50'	40'	6'-0"	W27x84	W24x68	W24x68	24K7	5'-0"	40G10N10.1K	69 ±
50'	50'	6'-0"	W30x90	W24x76	W24x84	26K10	5'-0"	48G10N13K	77 ±

LRFD Cantilever & Suspended Roof Girder System

Live Load = 30 psf
Dead Load = 18 psf
Wind Uplift = 14 psf



Exterior Column **Interior Column** **Girder Splice Plates**

Girder Span	Joist Span	Splice Dim.	Splice Plates - A36 Mat'l - (1 Near Side & 1 Far Side) 3/4"ϕ A325N Bolts		Connection at Exterior Column A36 - Dbl. Angle / 3/4"ϕ A235N Bolts	
"A"	"B"	"C"	Plate Size t x H x W	Bolts #/Rows @ 3" o.c.	Double Angles Thickness x Height	Bolts #/Rows @ 3" o.c.
30'	30'	6'-0"	5/16" x 7" x 0'-9"	2	1/4" x 0'-8 1/2"	3
30'	40'	6'-0"	5/16" x 10" x 0'-9"	3	1/4" x 0'-11 1/2"	4
40'	30'	6'-0"	5/16" x 10" x 0'-9"	3	1/4" x 0'-11 1/2"	4
40'	40'	6'-0"	5/16" x 10" x 0'-9"	3	1/4" x 0'-11 1/2"	4
40'	50'	6'-0"	5/16" x 10" x 0'-9"	3	1/4" x 0'-11 1/2"	4
50'	40'	6'-0"	5/16" x 10" x 0'-9"	3	1/4" x 0'-11 1/2"	4
50'	50'	6'-0"	5/16" x 13" x 0'-9"	4	1/4" x 0'-11 1/2"	4

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

JUNE

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

JUNE

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

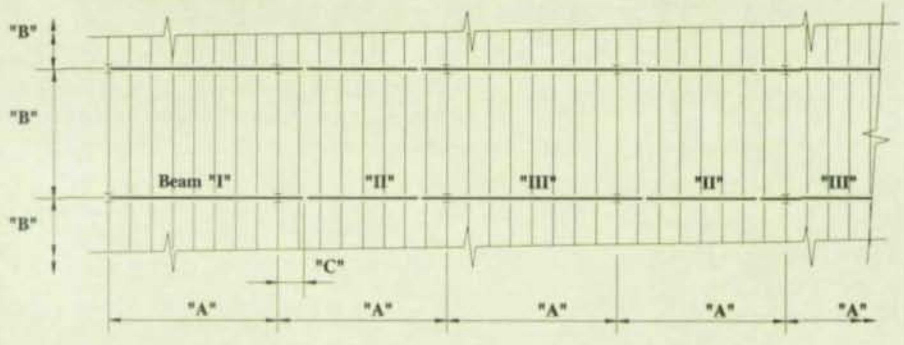
MODERN STEEL CONSTRUCTION

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

MODERN STEEL CONSTRUCTION
Reader Service Card

LRFD Cantilever & Suspended Roof Girder System

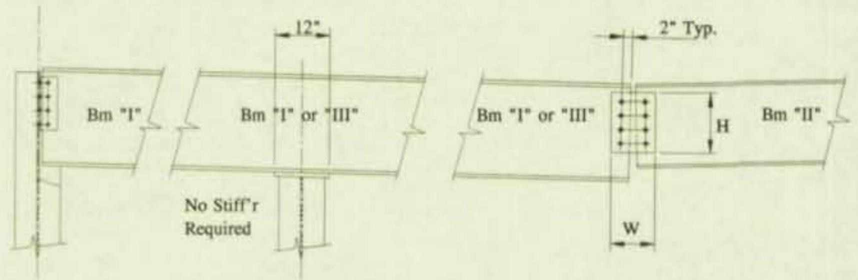
Live Load = 30 psf
Dead Load = 35 psf



Girder Span "A"	Joist Span "B"	Splice Dim. "C"	Recommended Wide Flange Member Design (50 ksi) Beam "I"	Beam "II"	Beam "III"	Joist Selection Designation	Spacing	Joist Girder Required Simple Span Condition Designation	Lbs./Ft.
30'	30'	6'-0"	W18x35	W16x31	W18x35	24K4	5'-0"	28G6N10K	35 ±
30'	40'	6'-0"	W21x44	W18x35	W18x40	26K9	5'-0"	32G6N13.2K	42 ±
40'	30'	6'-0"	W24x62	W21x44	W21x44	24K4	5'-0"	32G8N10K	51 ±
40'	40'	6'-0"	W24x68	W21x50	W24x55	26K9	5'-0"	32G8N13.2K	66 ±
40'	50'	6'-0"	W27x84	W24x62	W24x68	32LH6	5'-0"	40G8N16.4K	77 ±
50'	40'	6'-0"	W30x99	W24x76	W27X84	26K9	5'-0"	52G10N13.2K	75 ±
50'	50'	6'-0"	W30x116	W27x84	W30x99	32LH6	5'-0"	60G10N16.4K	79 ±

LRFD Cantilever & Suspended Roof Girder System

Live Load = 30 psf
Dead Load = 35 psf



Exterior Column **Interior Column** **Girder Splice Plates**

Girder Span "A"	Joist Span "B"	Splice Dim. "C"	Splice Plates - A36 Mat'l - (1 Near Side & 1 Far Side) 3/4"ϕ A325N Bolts — Plate Size t x H x W	Bolts #/Rows @ 3" o.c.	Connection at Exterior Column A36 - Dbl. Angle / 3/4"ϕ A235N Bolts — Double Angles Thickness x Height	Bolts #/Rows @ 3" o.c.
30'	30'	6'-0"	5/16" x 7" x 0'-9"	2	1/4" x 0'-8 1/2"	3
30'	40'	6'-0"	5/16" x 10" x 0'-9"	3	1/4" x 0'-8 1/2"	3
40'	30'	6'-0"	5/16" x 10" x 0'-9"	3	1/4" x 0'-11 1/2"	4
40'	40'	6'-0"	5/16" x 10" x 0'-9"	3	1/4" x 0'-11 1/2"	4
40'	50'	6'-0"	5/16" x 10" x 0'-9"	3	1/4" x 0'-11 1/2"	4
50'	40'	6'-0"	3/8" x 13" x 0'-9"	4	5/16" x 1'-2 1/2"	5
50'	50'	6'-0"	3/8" x 13" x 0'-9"	4	5/16" x 1'-2 1/2"	5

LRFD Cantilever & Suspended Roof Girder System

Design Examples

Member Design

Given:

Girder span	= 40 ft.
Joist span	= 30 ft. @ 5 ft. spacing
Fy	= 50 ksi
Live load	= 12 psf
Dead load	= 18 psf
Wind uplift	= 14 psf

Solution:

Calculate factored loading:

Live load	= 12 x 5	= 60 plf
Dead load	= 18 x 5	= 90 plf
Wind Uplift	= 14 x 5	= 70 plf

Min. dead load (excl. HVAC, Elec. etc.)
= 8 x 5 = 40 plf

(self-weight of girder included in the computer analysis)

Load Combination *I (1.2D + 1.6L)*:
LRFD Spec. Sect A4.1)

Factored Loading = 1.2(90) + 1.6(60)
= 204 plf on joists

Point loads on girder = 0.0055 x 30
= 6.12 kips ↓

Load Combination II (.9D + 1.3W):

Factored Loading = .9(40) - 1.3(70)
= -55 plf

Point loads on girder = 0.055 x 30
= 1.65 kips ↑

Member I Design:

Load Combination I

From computer analysis, $+M_{u,max}$ = 207 kip-ft., L_u = 5 ft. and $-M_{u,max}$ = 138 kip-ft., L_u = 6 ft.
From the *LRFD Manual Vol. I*, Load Factor Design Table, W18x35 has ϕMp = 249 kip-ft. with L_p = 4.3 ft., BF = 10.7 kips. By inspection, the positive moment will control. Find capacity of W18x35 for L_u = 5 ft.

ϕM_n = 249 - 10.7(5-4.3) = 241.5 kip-ft. > 207 kip-ft. **o.k.**

Total service load deflection exceeds *L*/240 for W18x35, therefore use W18x40.

Load Combination II

From computer analysis, $-M_{u,max}$ = 46.2 kips-ft., L_u = 35 ft.
From the Load Factor Design Selection Table, for W18x40, ϕM_p = 294 kip-ft., L_p = 4.5 ft., L_r = 12.1 ft. Since $L_u \geq L_r$, calculate ϕM_n from LRFD Spec. Eqn. (F1-13) with I_y = 19.1 in.^{4}, J = 0.81 in.^{4}, C_w = 1440 in.^{6}, assume C_b = 1:

$$\phi_b M_n = \frac{0.9\pi}{35x12}\sqrt{29000(19.1)(11200)(0.81) + [\frac{\pi 29000}{35x12}]^2(19.1)(1440)}$$
$$= 44.6 < 46.2 \text{ kip - ft.} \qquad \textbf{n.g.}$$

Use: W21x44 for Member 1

Member II Design:

Load Combination I

From computer analysis, $+M_{u,max}$ = 138 kip-ft. and L_u = 5 ft. From Load Factor Design Selection Table, the W12x26 with ϕM_p = 140 kip-ft. and L_p = 5.3 ft. is **o.k.**

Total service load deflection exceeds *L*/240 for W12x26 and W16x16, therefore use W16x31.

Load Combination II

From computer analysis, $-M_{u,max}$ = 30.8 kip-ft. and L_u = 28 ft. > L_r = 11.0 ft. for W16x31. Check W16x31 using LRFD Spec. Eqn. (F1-13): $\phi_b M_n$ = 35.2 > 30.8 kip-ft. **o.k.**

Use: W16x31 for Member II

Member III Design:

Load Combination I

From computer analysis, $+M_{u,max}$ = 138 Kip-ft., L_u = 5 ft. and $-M_{u,max}$ = 138 kip-ft., L_u = 8.33 ft. From the Load Factor Design Selection Table, for a W14x30, $\phi_b M_p$ = 177 kip-ft., L_p = 5.3 ft., L_r = 13.7 ft. and *BF* = 6.06 kips. Negative moment controls and $Lp < 8.33 < Lr$, therefore,

ϕM_n = 177 - 6.06(8.33 - 5.3) = 158.6 > 138 kip-ft. **o.k.**

Total service load deflection exceeds L/240 for W14x30, therefore use W16x31.

Load Combination II

From computer analysis, $-M_{u,max}$ = 30.8 kip-ft., L_u = 28.75 ft. From Member II Design, $\phi_b M_n$ = 35.2 kip-ft. > 30.8 **o.k.**

Use: W16x31 for Member III

Splice Connection Design

Given:

l_v = 2 in. — on all elements
l_h = 2 in. — on all elements
1 in. maximum between member ends
$^5/_{16}$ in. minimum plate thickness
ASTM A36 plate material, F_y = 36 ksi, F_u = 58 ksi
3 in. bolt spacing
$^3/_4$ in ϕ A325N bolts
Minimum connection depth T/2 of connected members
Minimum 2 bolt connection
Bolted/bolted design
R_u = 16 kips (from computer analysis)
W21x44 cantilevered member
W16x31 suspended member

Solution:

Check Bolts:
Minimum connection plate depth to meet T/2 criteria = $9^1/_8$ in. ± minimum 3 bolt connection.

ϕr_v = 31.8 kips/bolt, double shear
Eccentrically Loaded Bolt Group (LRFD Manual, Volume II, Table 8-18):
e_x = e/2 = 5/2 = 2.5 in.
n = 3 (first trial)
C = 1.99
ϕR_n = 1.99 x 31.8 = 63.28 >> 16 kips **o.k.**

Bearing on W16x31 web material (LRFD Manual, Volume II, Table 8-13):
ϕR_n = C x (2.4dtF_u) = 1.99 x (2.4 x 0.75 x 0.275 x 65)
ϕR_n = 64.03 kips > 16 kips **o.k.**

Shear on W16x31 LRFD Specification, Equation F2-1):
ϕR_n = $\phi 0.6F_yA_w$ = 0.90 x 0.6 x 50 x (15.88 x0.275)
ϕR_n = 117.9 kips >> 16 kips **o.k.**

Net shear on splice plates (LRFD Specification, Equation J4-1):
ϕR_n = $\phi 0.6F_uA_n$ = 0.75 x 0.6 x 58 x ((10-(3 x .8125)) x 2 x .3125
ϕR_n = 123.36 kips >> 16 kips **o.k.**

Gross shear on splice plates (LRFD Specification, Equation F2-1):
ϕR_n = $\phi 0.6F_yA_g$ = 0.90 x 0.6 x 36 x (10 x 2 x .3125)
ϕR_n = 121.50 kips >> 16 kips **o.k.**

Block shear rupture on splice plates (LRFD Manual, Volume II, Table 8-47a,b & Table 8-48a,b):
Table 8-47a coefficient = 68
Table 8-47b coefficient = 137
ϕR_n = (68 + 137) x 2 x 0.3125 = 128.13 kips

Table 8-48a coefficient = 152
Table 8-48b coefficient = 54
ϕR_n = (152 + 54) x 2 x 0.3125 = 128.75 kips **o.k.**

Flexural yield on splic plates (LRFD Specification, Chapter F):
M_u = R_ue/2 = 2.5 x 16 = 40 kip-in.
S_x = (t x H^2)/6 = (100 x 2 x .3125)/6 = 10.42 in^3
ϕM_n = ϕF_yS_x = 0.90 x 36 x 10.42
ϕM_n = 33.50 kip-in. >> 40 kip-in. **o.k.**

Flexural rupture on splice plates (LRFD Manual, Volume II, Table 12-1):
S_n = 6.25 in.3 from Table 12-1 (conservative by H = 9 in. in table)
ϕM_n = ϕF_uS_n = 0.75 x 58 x 6.25
ϕM_n = 271.87 kip-in. >> 40 kip-in. **o.k.**

Double Angle Connection at Exterior Column

Given:

l_v = $1^1/_4$ in.—on connecting angles
l_h = $1^1/_2$ in.—minimum
$^1/_4$ in. minimum connection angles
ASTM A36 plate material, F_y = 36 ksi, F_u = 58 ksi
3 in. bolt spacing
$^3/_4$ in. ϕ A325N bolts
Minimum connection depth T/2 of connected members
Minimum 2 bolt connection
Bolted / Bolted design
R_u = 25 kips (from computer analysis)
W21x44 member

Solution:

Double Angles (LRFD Manual, Volume II, Table 9-2):
$11^1/_2$ in. long connection required to meet T/2 criteria
2-$^1/_4$ in. angles with 4 rows $^3/_4$ in. ϕ A325N bolts
ϕR_n = 104 kips >> 25 kips **o.k.**

Shear on W21x44 (LRFD Specification, Equation F-2):
ϕR_n = $\phi 0.6F_yA_w$ = 0.90 x 0.65 x 50 x (20.66 x 0.350)
ϕR_n = 195.23 kips >> 25 kips **o.k.**

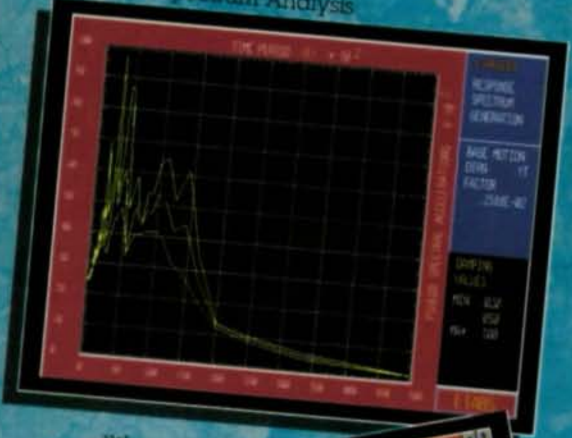
Bearing on W21x44 web material (LRFD Manual, Volume II, Table 8-13)
ϕR_n = 87.8 x t x n = 87.8 x 0.35 x 4
ϕR_n = 122.92 kips > 25 kips **o.k.**

STATE of the ART
Structural Engineering Software

EARTHQUAKE ENGINEERING

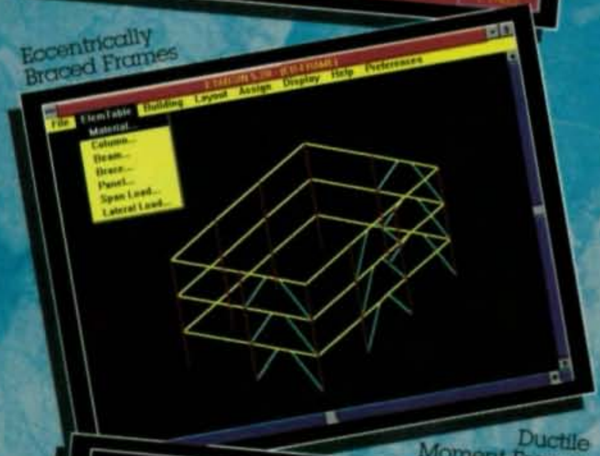
Integrated Seismic Analysis & Design of Concrete & Steel Structures
Based Upon 20 Years of Pioneering Research & Development
Developed by Edward L. Wilson & Ashraf Habibullah

Response Spectrum Analysis

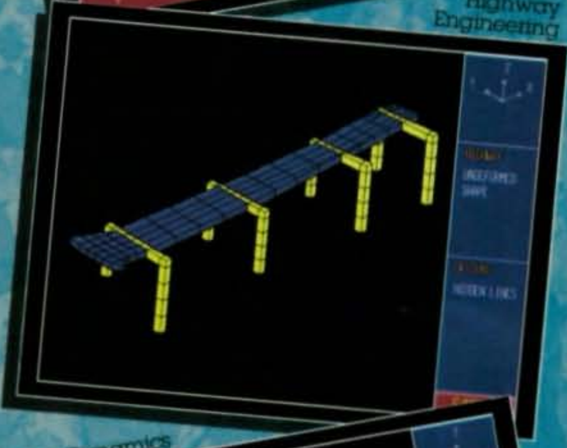


Time History Analysis

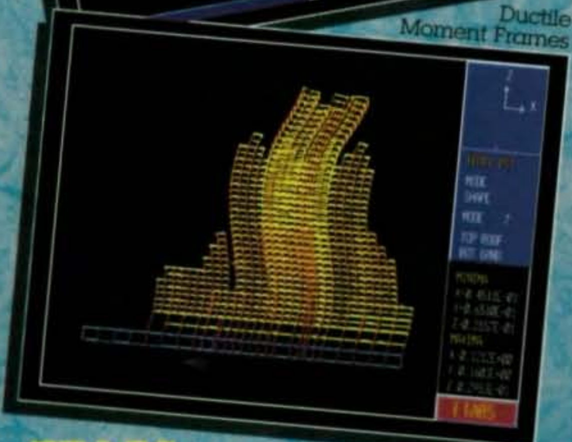
Eccentrically Braced Frames



Highway Engineering



Ductile Moment Frames



Bridge Dynamics

ETABS®
Building Analysis & Design

SAP90™
General Analysis & Design

CSI COMPUTERS & STRUCTURES INC.

For more information:
Computers & Structures, Inc.
1995 University Avenue
Berkeley, California 94704
TEL: (510) 845-2177
FAX: (510) 845-4096