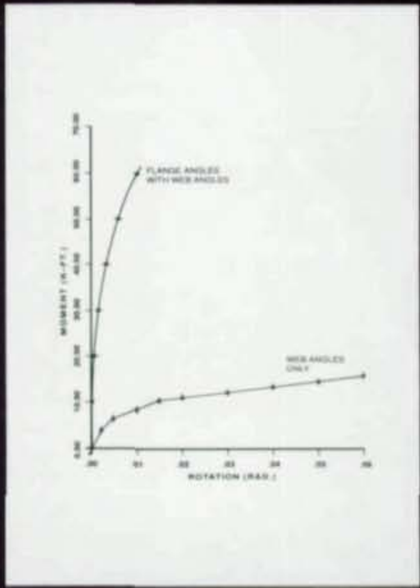
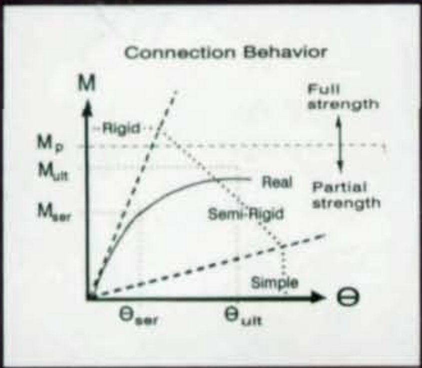
MODERN STEEL CONSTRUCTION

October 1992 $3.00



Special Report



Partially Restrained Connections

UNITED STEEL DECK, INC. DECK DESIGN DATA SHEET No. 18

TENSILE STRENGTH OF ARC PUDDLE WELDS - Wind Uplift Forces on Roof Deck

Steel	Ga.	(1) Visible Weld dia. .5	(1) .625	(1) .75	(1) 1.0	(2) Visible Weld dia. .5	(2) .625	(2) .75	(2) 1.0	(3) Visible Weld dia. .5	(3) .625	(3) .75	(3) 1.0
A446 grade A*	22	230	300	360	480	440	560	680	930	160	210	250	340
Fy = 33 ksi	20	280	350	430	580	520	670	820	1120	200	250	300	410
Fu = 45 ksi	18	360	460	560	760	650	840	1040	1440	250	320	390	530
	16	440	570	690	940	730	1020	1270	1770	310	400	490	660
A446 grade C	22	280	360	440	590	530	690	840	1140	200	250	310	410
Fy = 40 ksi	20	340	430	520	710	630	810	1000	1360	240	300	370	500
Fu = 55 ksi	18	440	560	680	930	790	1030	1280	1760	310	390	480	650
	16	540	690	850	1150	730	1240	1550	2160	380	490	590	810
A446 grade D	22	310	390	480	640	580	750	910	1240	220	280	330	450
Fy = 50 ksi	20	370	470	570	770	690	890	1090	1490	260	330	400	540
Fu = 60 ksi	18	480	610	750	1010	860	1130	1390	1920	340	430	520	710
	16	590	760	920	1260	730	1350	1690	2360	410	530	650	880
A611 grade C*	22	250	310	380	510	470	600	730	990	170	220	270	360
Fy = 33 ksi	20	300	380	460	620	550	710	870	1190	210	260	320	430
Fu = 48 ksi	18	380	490	600	810	690	900	1110	1540	270	340	420	570
	16	470	610	740	1010	730	1080	1350	1890	330	420	520	710
A611 grade D	22	270	340	410	560	500	650	790	1080	190	240	290	390
Fy = 40 ksi	20	320	410	500	670	600	770	940	1290	230	290	350	470
Fu = 52 ksi	18	420	530	650	880	750	980	1210	1670	290	370	450	610
	16	510	660	800	1090	730	1170	1460	2040	360	460	560	760

* Roof deck is generally specified to meet ASTM A446 grade A (galvanized) or A611 grade C (painted).

(1) Single metal thickness values. (2) Double metal thickness values - end laps (3) Edge laps (at supports).

All table values are in pounds (tension) and are **design** values found by the formulas given in the AISI Specification for the Design of Cold-Formed Structural Members[(A)]; the safety factor is 2.5 but the 33% increase for wind loading has been included. The edge lap values (column 3) have been reduced by 30% to adjust for eccentric loading of the weld as recommended by Tensile Strength of Welded Connections[(B)]. AWS procedures for arc puddle welds are to be followed. A minimum electrode strength of 60 ksi is required.

(A) August 19, 1986, Edition with December 11, 1989 Addendum.

(B) R.A. LaBoube and Wei-Wen Yu, Department of Civil Engineering, Center for Cold Formed Steel Structures, University of Missouri--Rolla, Civil Engineering Study 91-3

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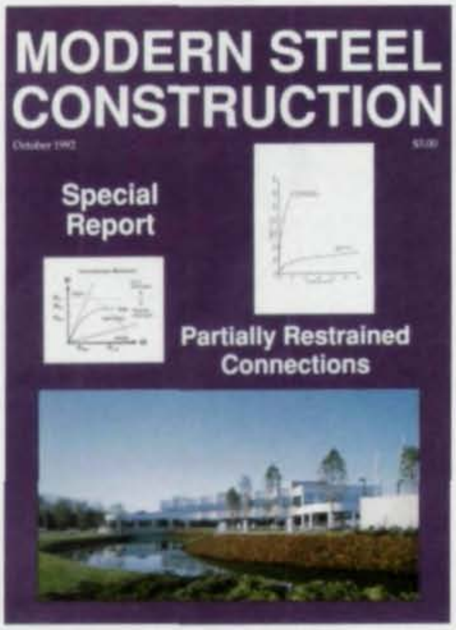
MODERN STEEL CONSTRUCTION

Volume 32, Number 9 September 1992



The General Accident Insurance (Nashville Branch) building was designed using PR connections. Cost reductions were realized through the elimination of field welded connections, elimination of column web stiffeners, reduction of frame erection schedule and the simplification of beam-column connections. A Special Report on PR connections begins on page 18.

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FEATURES

NEWS AND DEPARTMENTS

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Revisiting An Old Friend

In my first editorial for *Modern Steel Construction* way back in January 1990, I mentioned that AISC was moving its headquarters to a very suitable building—at least for a structural steel association. "Whether by chance or some pre-ordained plan," I wrote, "we ended up in the first multi-story building to be built with high strength steel with a yield point of 50,000 psi furnished to ASTM specification A440."

I also quoted from a USS Structural report on the building that noted the building used moment resisting beam-to-beam connections instead of diagonal bracing. "Because wind forces require moment resisting beam-to-column connections, it was considered advisable to include this restraining effect in the design of the beams for gravity loads." In other words, the building was an early example of Partially Restrained Connections—the topic of a Special Report in this issue beginning on page 18.

In 1962, the engineer designed his innovative structure to achieve *minimal interference on leasing space from structural members.* Also, the use of high-strength steel in conjunction with PR Connections allowed a substantial savings in steel weight.

Today, higher fabrication labor costs further increase the advantages associated with PR connections. These include the elimination of column web stiffeners and the simplification of beam-column connections. In addition, frame erection time is quickened and the reduced weight of the frame can reduce foundation costs.

Professor Roberto Leon from the University of Minnesota begins our special report with an explanation of the principles involved in designing PR connections. His article is followed by one from Kurt Swenson, an engineer with Stanley D. Lindsey & Associates. Kurt describes his firm's use of PR connections and sites the cost effective design used on a recently completed low-rise building in Nashville.

If you've used PR Connections in a recent design—or, for that matter, in an old building like One East Wacker Drive—let me know. We're always looking for innovative steel stories to cover in *MSC*. **SM**

AISC Software

AISC Database

The AISC Database contains properties and dimensions of structural steel shapes, corresponding to Part 1 of the 1st edition *LRFD Manual of Steel Construction* and the 9th edition *ASD Manual of Steel Construction*. Two versions, one in U.S. customary units and one in metric units, are available. Please specify.

The computer database, in ASCII format, contains W, S, M, and HP shapes, American Standard Channels (C), Miscellaneous Channels (MC), Structural Tees cut from W, M, and S shapes (WT, MT, ST), Single and Double Angles, Structural Tubing, and Pipe

An explanation of variables specified in each data field is included as are a BASIC read/write program and a sample search routine by which the database may be manipulated, and a routine to convert the file to Lotus 1-2-3 format. Additionally, the metric version includes a text file which cross references the ASTM designations in SI units to U.S. customary units.

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CONXPRT is a knowledge based PC software system for steel connections. Expert advice from long-time fabricator engineers is used to augment the design rules. CONXPRT incorporates provisions to set dimensional and material defaults for a particular project or general shop needs. Additionally, CONXPRT is menu driven and incorporates help screens designed for easy use.

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Available in either 1st edition LRFD or 9th edition ASD format. Designs more than 80 configurations of double framing angles, shear end plates, and single plate shear connections is possible.

Module II: Moment Connections

Available in 9th edition ASD format only. Provides a set of four knowledge bases for the design of strong axis moment beam-to-column flange connections; direct welded, flange welded-web bolted, flange plate welded-web bolted, and flange plate bolted-web plate bolted connections. Additionally, a knowledge base for the column side design of web stiffener plates and doubler plates is a part of the module.

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Module I ASD & Module I LRFD ***$550.00***

Module II ASD ***$400.00***

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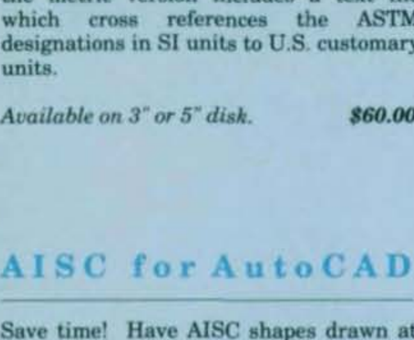
Save time! Have AISC shapes drawn at your command with AISC for AutoCAD (Release 10 and above). It draws shapes and lists properties corresponding to data published in Part 1 of the 1st edition *LRFD Manual of Steel Construction* and the 9th edition *ASD Manual of Steel Construction*.

The program will parametrically draw to full scale the end, elevation, and plan views using the design dimensions of the shapes shown below.



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STEMFIRE

STEMFIRE determines safe and economical fire protection for steel beams, columns, and trusses. STEMFIRE is based on rational procedures developed by AISI which extend the published UL fire resistive designs to other possible rolled structural shapes and common protection material requirements. For a required fire rating, STEMFIRE determines the minimum spray-on thickness for various rolled shapes as well as the ceiling membrane or envelope protection for trusses. This methodology is recognized by UL and has been adopted by the three national model building codes in the USA.

The software database contains all pertinent steel shapes and many listed UL *Fire Resistance Directory* construction details and fire ratings. In this manner, user search time is minimized and the design and checking of steel fire protection is optimized. Hence, STEMFIRE is easy to use with little input effort to quickly produce specific design recommendations. Two 5" disks containing executable software bearing AISC copyright and a users manual with instructions and sample problems are included with purchase.

Available on 5" disk only. ***$96.00***

WEBOPEN

WEBOPEN is designed to enable engineers to quickly and economically design beam web openings. An expedient tool, WEBOPEN uses state of the art criteria and features a clear and logical data entry system with easy to use color coded input windows. Furthermore, WEBOPEN accesses a shape database allowing the selection of any W, S or M shape for use in the design procedure.

WEBOPEN was written by practicing engineers for engineers and incorporates expert design checks and warning messages which enhance the application of the AISC Design Guide to specific design problems. Using this software, unreinforced or reinforced, rectangular or round openings, concentric or eccentric, in both composite and non-composite steel beams may be designed. The design is complete with stability and proportioning checks. Additionally, the design is optimized through user interaction during the design sequence. Included with purchase are the WEBOPEN program, the WEBOPEN Users Manual and the AISC Design Guide *Steel and Composite Beams with Web Openings*

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Steel Interchange

Steel Interchange is an open forum for *Modern Steel Construction* readers to exchange useful and practical professional ideas and information on all phases of steel building and bridge construction. Opinions and suggestions are welcome on any subject covered in this magazine. If you have a question or problem that your fellow readers might help 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
1 East Wacker Dr.
Suite 3100
Chicago, IL 60601

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. It is recognized that the design of structures is within the scope and expertise of a competent licensed structural engineer, architect or other licensed professional for the application of principles to a particular structure.

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

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

How should I connect wide flange beams to all four faces of a structural tube column in such a way as to transfer wind moments as well as dead and live load reactions?

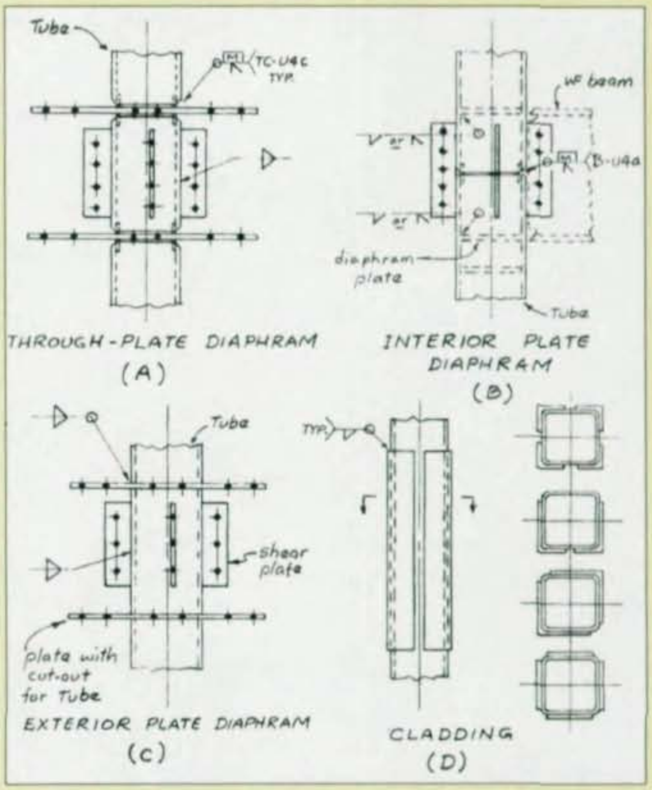
It must be assumed that the tube column is able to resist the bending induced by the various combinations of wind and gravity moment. The tube walls must be protected against localized buckling and stretching and the side walls must have adequate shear capacity, unless the horizontal forces are able to be carried in one face, directly through, and out the opposite face. There are several ways to strengthen a tube column.

1. Internal diaphragm
2. External diaphragm
3. Girdling or cladding
4. Through plate diaphragm.

When all four beams are the same nominal depth, through-plate diaphragms can be used. Figure A is and example of this, showing the tube severed and rewelded to the plates. Another version is shown in Figure C where external diaphragm plates are cut out to the profile of the tube and welded to the tube. The tube remains intact. Figure B shows and example of internal diaphragms. The tube is cut and the diaphragm plates installed where required and the tube rewelded. This is useful if the wide flange beams are of varying nominal depths. In Figure B the moment connection is made by field welding the beam flange directly to the face of the column. The fourth method of reinforcing the tube is by girdling or cladding as shown in Figure D. By extending the reinforcing upward and downward more bending strength can be added. Resistance to shear in the sidewalls can also be increased by girdling.

The gravity load can be resisted by a shear plate or single angle connection or, as noted in Figure E, a stiffener plate can be installed below the bottom flange connection to connect it to a stiffened seat.

Figure F shows several adaptations of external diaphragm plates and their versatility.



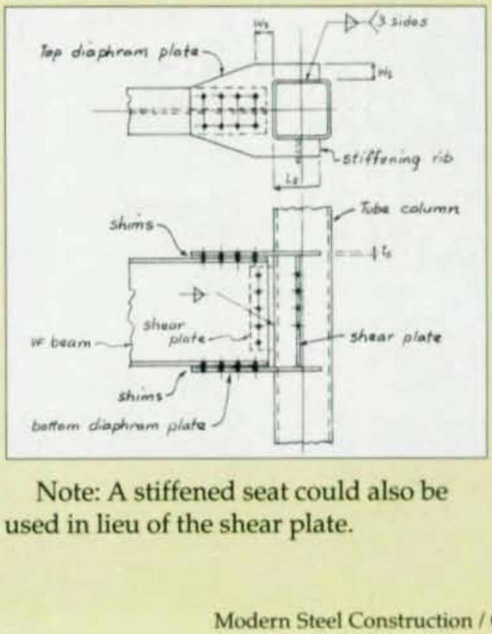


Figure E

Note: A stiffened seat could also be used in lieu of the shear plate.

Steel Interchange

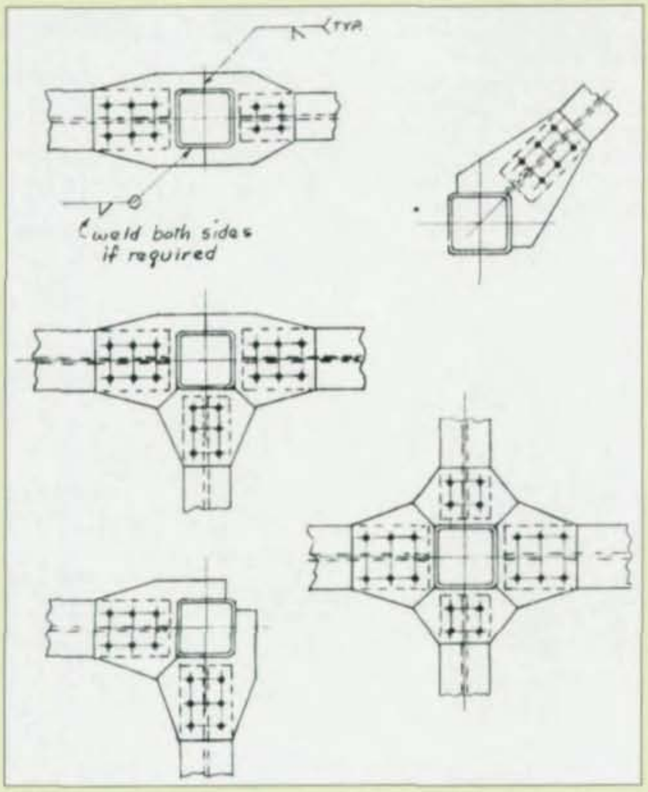


Figure F: Adaptations of external diaphrams

This is but a brief glimpse at a complex type of connection. For further related information I suggest the following references:

1. White, Richard N., "Framing Connection for Square and Rectangular Structural Tubing", AISC Engineering Journal, July 1965.

2. Nippon Steel Metal Products, Inc., "Design Manual of Structural Tubing - Square and Rectangular", 1977.

3. Stelco, Inc., "Hollow Structural Section - Design Manual of Connections", 2nd Edition 1981, Stelco, Inc., Hamilton, Ontario, Canada.

4. Ricker David T., "Comments on the Behavior of Moment Connection of Wideflange Beams to Tube Columns", Address at Structural Steel Fabricators of New England Spring Symposium, Worcester Polytechnic Institute, 1985.

5. Ricker, David T., "Practical Tubular Connections", 1985 ASCE Structural Engineering Conference

David T. Ricker
Payson, AZ

Another response:

A simple connection is suggested. Details are provided in Figure 1.

Vijay P. Khasat
Ohio Edison
Akron, OH

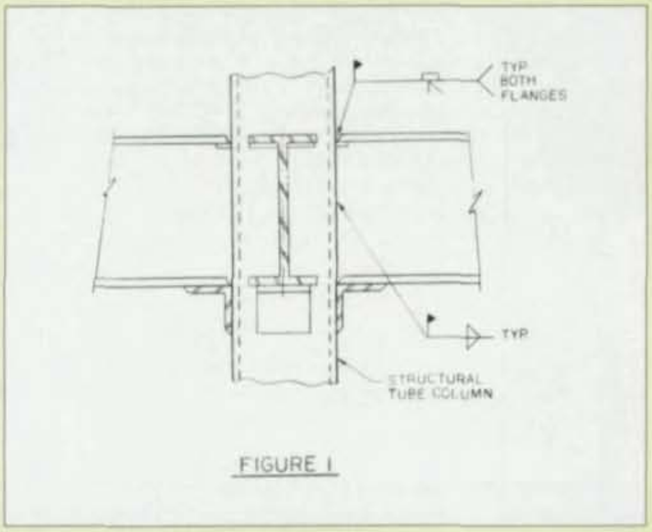


FIGURE 1

Are there any design requirements that an engineer can follow when designing lateral bracing?

When designing lateral bracing the engineer has little in the way of guidance from the AISC Specifications although the commentary suggests referencing the Structural Stability Research Council's *Guide to Design Criteria for Metal Compression Members* (Wiley Interscience , New York, ISBN 0 471-09737-3). In addition the *Handbook of Steel Construction* published by the Canadian Institute of Steel Construction is more specific and does provide design requirements in its section 20.3 Stability of Beams, Girders and Trusses.

Frank Petrigliano
The Steel Institute of New York
New York, NY

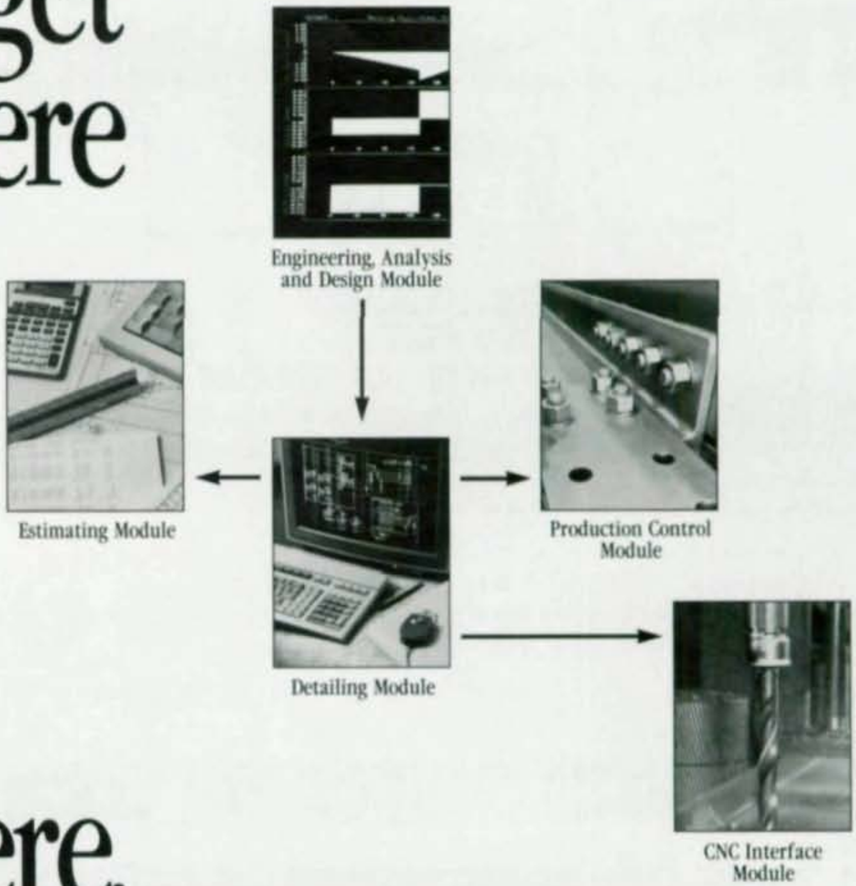
New Questions

Listed below are some 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. 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.

1. Where can I get information on stainless steel bolts?

2. Are there limits on bending a wide flange beam into a radius?

How to get from here



to here.

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CORRESPONDENCE

Extra Credit

Dear Editor:

I am in receipt of the August 1992 issue of *Modern Steel Construction*. While I was happy to find our article on the Old Bridge Public Library, I was horrified to find that the list of Design Team Members had been omitted.

The average reader will conclude that my role in this project was much more substantial than was, in fact, the case. You have done a terrible disservice to both our Project Architect, Michael DeBiasse, and to the Project Engineer, Sanjeev Shah, of Severud Associates.

These two individuals deserve the lion's share of the credit for the realization of this project, and you have an obligation to recognize their contributions.

Sincerely,
Eliot W. Goldstein, AIA
James Goldstein & Partners
Millburn, NJ

Cellular Beams

Dear Editor:

Terminal 1 at Chicago's O'Hare Airport, recently featured in you publication (June 1992), is a famous example of architecture using exposed steel beams with openings or apertures profiled along the web. Westok Structural Services, Ltd., based in Great Britain, has developed a technique that produces such beams with the maximum economy. Instead of simply burning disks of steel out of the web, which is both wasteful and costly, Westok's method uses a patented cutting profile similar to that used in the production of castellated beams.

The profiled tee sections are welded back together to form a beam of increased depth, thus significantly improving the beams overall strength and performance. In the case of O'Hare Airport, for example, Cellform beams would have been typically 30% lighter

than the sections actually used.

O'Hare, like all other airports, could also make use another—and perhaps more important—application of Cellform beams. In airport terminals, the necessity for long clear spans without intermediate columns is a constant headache for structural engineers. The excellent strength/weight ratio of a Cellform beam makes it a most efficient long-span floor beam, particularly effective when designed compositely. The fact that air-conditioning ducts and other services can be passed through the cells ensure the shallowest of overall floor considerations, considerably reducing exterior cladding costs.

A final refinement can be achieved by producing an asymmetrical Cellform beam made from two tees of different mass. The lighter steel is used as the compression flange—under used in composite design—resulting in the lightest and most efficient use of steel. Again, Cellform floor beams are typically 30% lighter than solid web beams, producing construction depth reductions of up to 19" per floor.

Although not yet used in the U.S., Cellform beams are not a new concept to American architects. One example is HOK International, Ltd., whose London office has designed an Exhibition Hall incorporating more than 1,100 tons of

straight and curved Cellform roof beams spanning up to 90′.

Westok operates a free design service, already used by a number of consulting engineers in the States, and expects the first Cellform project to commence here inside of six months. Engineers wishing to consider cellular beams should contact: Andy Holmes, Westok Structural Services, Ltd., Horbury Junction Industrial Estate, Horbury Junction, Wakefield, West Yorkshire, UNITED KINGDOM WF4 5ER, Tel: 0924 264121; Fax: 0924 280030.

Sincerely,
Andy Holmes

New Ideas In Structural Steel

After a one year hiatus, AISC's Lecture Series on Structural Steel is returning in 1993

Fundamental changes in the economic and business climate are shifting the priorities of designers and fabricators. AISC's new Lecture Series, "New Ideas In Structural Steel," will cover four topics, each reflective of the most up-to-date industry practices, at each lecture.

The first subject broached is the increased emphasis on low-rise construction. While not as glamorous as designing a 100-story landmark, these structures still offer unique challenges. This part of the session utilizes *AISC Steel Design Guide #5, Low- and Medium-Rise Buildings* as an introduction to efficient structural steel systems for low-rise structures. Topics include live load and bay size, composite floors, vibration, wind loads, unbraced frames, and special techniques.

Next up is the new *Manual of Steel Construction, Volume II.* This manual is devoted exclusively to connections, and this part of the session will highlight pertinent information relative to understanding the efficient selection of connection types and critical design parameters. Particular emphasis will be placed on side-by-side comparisons of Allowable Stress Design (ASD) and Load and Resistance Factor Design (LRFD) solutions.

The third subject covered in this new lecture series is Eccentrically Braced Frames. This steel innovation was created to provide a predictable and economic system for large seismic forces. But its inherent economy has also proven to be valuable in moderate seismic or even wind loadings in low-rise buildings. In addition, EBF relieves the interference problems of concentrically braced frames, while at the same time providing new opportunities for architectural expression.

The final topic to be covered is Partially Restrained Connections. This is an old idea whose time has come. For years, the idea of a more economical framing method using the principle of connection flexibility to provide more balanced designs has been the subject of much research and analysis. Now, computers have tamed the additional analytical steps to bring new, reliable information to the designer. Fundamental principles will be covered, including proposed methods.

Lectures are tentatively scheduled for the following cities: Southwest—Dallas, Houston, San Antonio, Kansas City, New Orleans, Denver;

Midwest—St. Louis, Chicago, Detroit, Minneapolis, Indianapolis;

Mid-Atlantic—Baltimore, Washington DC, Philadelphia, Cleveland, Cincinnati, New York, Newark;

Northeast—Meriden, Boston, Albany, Rochester;

Southeast—Orlando, Atlanta, Birmingham, Charlotte, Raleigh, Greenville, Miami, Knoxville, Columbia, Wilmington;

West: San Diego, Irvine, San Francisco, Seattle, Los Angeles.

In addition, approximately 10 other cities are under consideration.

For more information, write: AISC Lecture Series, One East Wacker Dr., Suite 3100, Chicago, IL 60601-2001.

1992 AISC Seismic Provisions

The new June 15, 1992 *AISC Seismic Provisions for Structural Steel Buildings* is now available. It contains seismic steel detailing provisions that supplement the main *AISC Load and Resistance Factor Design* and *Allowable Stress Design Specifications.* As with the previous edition, requirements for columns, ordinary moment frames, special moment frames, concentrically braced frames and eccentrically braced frames are given.

The principle changes are the conversion to the load and design format contained in the 1991 National Earthquake Hazards Reduction Program (NEHRP) *Recommended Provisions for the Development of Seismic Regulations for New Buildings.*

The new Seismic Provisions was developed by a subcommittee chaired by E.P. Popov, professor emeritus at the University of California (Berkeley) and with Clarkson W. Pinkham of S.B. Barnes & Associates serving as secretary.

Copies of the 1992 *AISC Seismic Provisions for Structural Steel Buildings* are available for $5.00 + $4.00 shipping and handling from: American Institute of Steel Construction, Inc., P.O. Box 806276, Chicago, IL 60680-4124 (312) 670-2400 ext. 433.

New LRFD Edition Nearing Completion

In accordance with the consensus operating procedures of the AISC Committee on Specification, a draft of the updated *AISC Load and Resistance Factor Design Specification and Commentary* is available from Oct. 1 through Nov. 30, 1992, for public review and comment. Copies of this draft (approximately 200 pages) are available for a nominal $10 handling charge from AISC Publications (312-670-2400 ext. 433).

Any public comments must be received by November 30, 1992, and should be as specific as possible. Once the AISC Committee on Specifications completes its work and approval, the new document is intended to replace the 1986 *LRFD Specification.*

Some of the principal changes include expansion of material on stability of unbraced frames, design provisions for slip critical bolted joints at factored loads, more general C_b equation, updated criteria for slender web girders and unsymmetric members, and new web crippling limits.

Corresponding corrective revisions to the 1989 AISC *ASD Specifications* also are being deliberated.

The annual meeting of the AISC Committee on Specification has been scheduled for Nov. 17-18, 1992 in the Chicago area. The meeting agenda primarily will address any new and unresolved specification issues. Anyone wishing to attend is welcome. Please contact N. Iwankiw, AISC Committee Secretary, at (312) 670-5415, to indicated your interest and to request more detailed information.

Steel Inspection Newsletter

Information on code changes, certification programs, and a variety of testing techniques is available in *Steel Inspection News: an informational digest*. The eight-page, bi-monthly newsletter is published by the Steel Structures Technology Center, Inc.

Topics include: structural steel materials; welding; bolting; nondestructive testing; steel bar joists; roof and floor decking; shear connectors; reinforcing steel; coating systems; tolerances and certification programs for inspectors and contractors.

A subscription costs $36 for one year, $60 for two years. For more information, contact: Steel Structures Technology Center, 40612 Village Oaks Dr., Novi, MI 48375-4462 (313) 344-2910; fax (313) 344-2911.

October 2. **Innovative Steel Design Concepts** and **Computers and Steel Design**, New York City. Contact: Steel Institute of New York (212) 697-5553.

October 5-6. **Central Fabricators Meeting**, Chicago. Fabrication and operations. Contact: LaVerne Duckrow, 7227 W. 127th St., Palos Hills, IL 60463 (708) 361-2332.

October 5-7. **Fabtech West Exposition and Conference**, Anaheim, CA. Contact: Society of Manufacturing Engineers, One SME Dr., P.O. Box 930, Dearborn, MI 48128 (313) 271-1500.

October 8. **The Rationale For The Use Of Composite Concrete And Steel Construction**, Chicago. Featured speaker is Cesar Pelli of Cesar Pelli and Associates. Contact: Sherwin Asrow, S.P. Asrow Associates, Ltd (312) 939-2150.

October 12. **Connections & Camber**, Albany. Breakfast meeting sponsored by NYSSFA. Contact: Brian Carmer (518) 695-3752.

October 12. Deadline for submission of papers for **Practical Solutions for Bridge Strengthening And Rehabilitation**, Des Moines. Contact: Bridge Engineering Center, Dept. of Civil and Construction Engineering, Iowa State University, Ames, IA 50011 (515) 294-8763; Fax (515) 294-8216.

October 13-14. **Welding Structural Design two-day seminar,** Houston. Designed to provide engineers and welding inspectors a greater understanding of weld mechanics and welded engineering structures. Contact: AWS, 550 N.W. LeJeune Road, P.O. Box 351040, Miami, FL 33135 (800) 443-9353.

October 13-14. **CADD Seminars**, Philadelphia. Topics include: implementation & management; developing CADD standards; automating the design office; A/E CADD management; linking computers; and training & staffing for CADD. Contact: Sharon Price, A/E/C Systems (800) 451-1196.

October 15. **Higgins Lecture Breakfast Meeting**, Kansas City. Featuring Prof. William Maguire. Contact: Gordon Finch (816) 373-1203.

October 15-16. **CADD Seminars**, San Francisco. (See October 13-14) Contact: Sharon Price, A/E/C Systems (800) 451-1196.

October 16. **Higgins Lecture at the 4th Annual Conference of Structural Engineers of Texas**, San Antonio. Featuring Prof. William Maguire. Contact: Jim Anders (214) 369-0664.

October 22. **Steel Bridge Forum,** Atlanta. Contact: Camille Rubeiz, Steel Bridge Forum, c/o AISI, 1101 17th St., N.W., Suite 1300, Washington, DC 20036 (202) 452-7190.

October 27. **Domestic Steel Industry Update**, breakfast meeting, Jackson, MS. Contact: Jim Anders (214) 369-0664.

October 28. **Addressing Issues in Connection Design: AISC's Manual of Steel Construction Volume II—Connections**, Columbus, OH. Breakfast meeting introducing a new AISC manual intended as a companion to both the ASD 9th Edition and LRFD 1st Edition manuals. Contact: Lloyd "Mac" Hughes (215) 431-1912.

November 2. **Structural Steel Concepts: Restraint Girder System**, New York City. A presentation by Neil Wexler, P.E., will describe his use of systems featuring precast concrete plank with composite steel girders, hung beam to girder connections, composite open web joist, and partially restrained connections. Contact: Steel Institute of New York (212) 697-5553.

November 10-11. **CADD Seminars**, Atlanta. (See October 13-14) Contact: Sharon Price, A/E/C Systems (800) 451-1196.

November 10-11. **Welding Structural Design two-day seminar,** Atlanta (See October 13-14 listing).

November 10-12. **Fourth Biennial Symposium on Movable Bridges**, Ft. Lauderdale, FL. More than 40 sessions on various aspects of moveable bridges. Contact: Andrew W. Herrmann at (904) 575-1970.

November 12-13. **CADD Seminars**, Chicago. (See October 13-14) Contact: Sharon Price, A/E/C Systems (800) 451-1196.

Tubular Sections In Building Construction. Breakfast meeting. Discussion includes: design criteria; Type 2 connections; tube-to-tube connections; design guides; practical recommendations; and application examples. Contact: Colleen Hays, AISC Marketing, Inc. (312) 670-2400 ext. 203.

Atlanta	October 8
Birmingham, AL	October 9
Charleston, SC	October 13
Columbia, SC	October 14
Cleveland	October 15
Wilmington, NC	October 15
Jackson, MS	October 17
Norfolk, VA	October 20
Jacksonville, FL	November 3
Orlando	November 4
Tampa	November 5
Cincinnati	November 10
Cocoa Beach, FL	November 17
New Orleans	November 17
Boca Raton, FL	November 18
Miami	November 19

Eccentric-Braced Frame Technology. Breakfast meeting. Discussion includes EBFs for both seismic and non-seismic applications. Contact: Colleen Hays, AISC Marketing, Inc. (312) 670-2400 ext. 203.

Columbia, SC	October 14
Charleston, SC	October 15
Wilmington, NC	October 16
Norfolk, VA	October 20
Richmond, VA	October 21
Wichita, KS	November 10
Denver	November 11
New Orleans	November 17

Structural Welding Design For Buildings. One day seminar discussing the design of shear, moment, splice and other structural connections, AISC and AWS specification requirements, and other welding issues. Contact: Robert E. Shaw, Jr., Steel Structures Technology Center, (313) 344-2910.

Chicago	October 8
Costa Mesa, CA	October 22
San Fransisco	October 28
New York City	December 3

Steel Buildings: Special Inspection. One-day seminar discussing code-required inspections of steel-framed buildings. Contact: Robert E. Shaw, Jr., Steel Structures Technology Center, (313) 344-2910.

Houston	October 1
Dallas	October 2
Chicago	October 9
Phoenix	October 19
Las Vegas	October 20
Costa Mesa, CA	October 21
City of Commerce, CA	October 23
Sacramento, CA	October 26
San Francisco	October 27
Portland	October 29
Seattle	October 30
Portland	November 30
Boston	December 1
Cromwell, CT	December 2
Hasbrouck Heights, NJ	December 4

Composite Semi-Rigid Connections

Partial Restraint connections can significantly reduce deflections, increase the frequency of vibration, and provide needed lateral stiffness

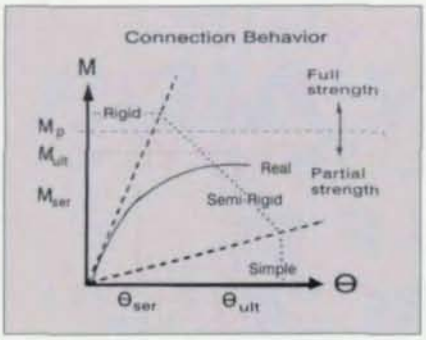


FIGURE 1: The behavior of a connection can be characterized by its moment-rotation curve, which shows the moment required to change the rotation between the beam and the column. The stiffness of the connection is given by the slope of the line, and can be characterized as rigid, semi-rigid, or simple. The connection strength can be either full strength (M>Mp) or partial strength. The ideal connection will have a high initial stiffness (at θser), and a limited strength at θult).

By Roberto Leon, Ph.D.

Recent steel design specifications, such as the LRFD Specification from AISC or Eurocodes 3 and 4, have adopted a limit state design approach, with ultimate strength and serviceability limit states generally being the governing criteria. While a design based on ultimate strength generally results in a very economical structure, as span lengths increase and new types of loading are added to the design, drift and deflection criteria begin to govern.

For the design of multistory steel structures, two recent developments have begun to pose problems:

- For the vast majority of low- and medium-rise buildings, composite floors have become the preferred structural system for gravity loads. However, since their design is most economical when based on ultimate strength limits, the economical span-to-depth ratios have increased on all-steel construction from about 20:1 to 24:1, and on composite construction from 28:1 to 32:1. The result is slender floors prone to problems with both short- and long-term deflection, cracking and vibrations.
- Large areas of the eastern and midwestern United States have been upgraded to seismic zones with design ground accelerations as high as 0.2g. In these areas, where lateral load design was traditionally governed by wind, new construction will have to comply with both more stringent drift criteria and some minimum level of seismic detailing to insure safety during an earthquake.

Both of these problems require that connections with additional stiffness, ductility and strength be incorporated into practice. Since connection fabrication and erection represent a significant portion of a structure's cost, neither the introduction of radically new connection technology nor a requirement for full restraint connections are practical since both would greatly increase costs.

The most logical solution, therefore, is to modify the connections commonly in use today to increase their strength and stiffness. Since partial restraint (PR, or Types 2 and 3) connections already possess some flexural strength and rigidity, it is logical to try to improve their performance rather than to promote a whole new technology. PR connections (Figure 1) can significantly reduce deflections, increase the frequency of vibration, and provide the required lateral stiffness for unbraced frames up to 10 stories.

Semi-Rigid Composite Connections

During the past eight years, the author and his co-workers at the University of Minnesota have developed the concept of semi-rigid composite connections. These connections utilize the additional strength and stiffness provided by the floor slab that is activated by adding shear studs and slab reinforcement in the negative moment regions adjacent to the columns. This type of connection is a very logical and economical solution to the problems described earlier.

Connection Investigations

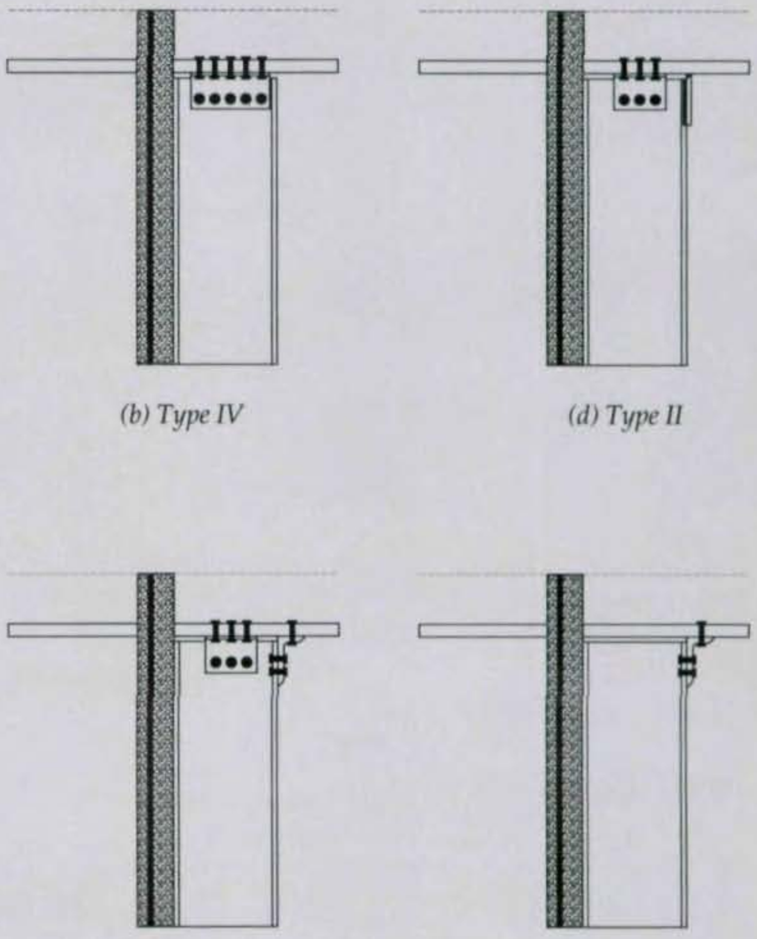
Four different types of connections have been investigated (Figure 2):

Type I: These consist of a typical seat angle, a double angle shear connection to the web, and continuous slab reinforcement across column lines (Figure 3). Under gravity loads the latter provides the tension part of the couple, while the angle in bearing acts as the compression member. Because of the increase in steel strength (Grade 60 for rebar vs. A36 or A572 for the angle) and lever arm, this connection adds significant moment capacity to the connection over a typical top and seat angle connection.

There also are significant stiffness gains because the slab steel yields in almost pure tension and at a higher stress than a top angle. Additional stiffness gains accrue from the use of slip critical bolts and, at large rotations, from the presence of web angles. Under seismic loading, however, the bottom angle will pull out at relatively low loads resulting in unsymmetrical and degrading hysteresis loops. Thus, for seismic applications the thickness of the bottom angle should be increased to yield a connection with more symmetrical moment-rotation characteristics. Figure 4 illustrates the possible increases in strength and stiffness that can be achieved and Figure 5 shows a full-scale, two-bay, one-story subassemblage incorporating these connections being tested under a combination of gravity and lateral loads.

Type II: These consist of a welded bottom plate, web angles for shear, and continuous slab reinforcement. This is a very stiff, economical connection possible because a welded plate has been substituted for the bottom angle, which in Type I connections was the weak link. This plate carries the tensile and compressive forces basically as axial loads, resulting in a very stiff and non-degrading connection. The same results can be achieved by welding the plate to the column in the shop and bolting it to the beam with high strength slip critical bolts in the field. The welds must be detailed to insure full transfer of moment and to eliminate the possibility of weld fracture. This connection offers very large initial stiffnesses and symmetrical behavior under cyclic loading.

Figure 2.



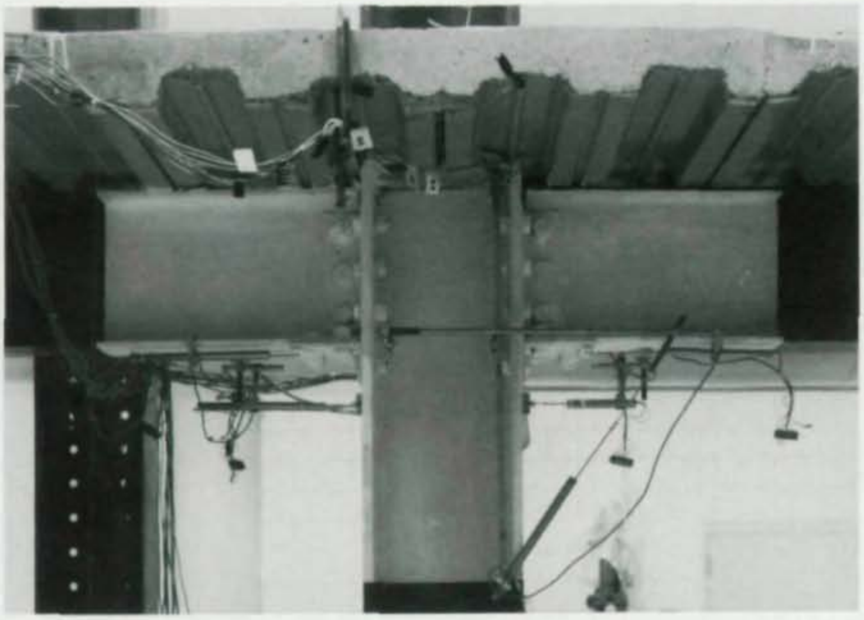
(b) Type IV — (d) Type II — (a) Type I — (c) Type III

Type III: These consist of connections similar to Type I, except that the web angles are missing. This results in a connection with a much flatter inelastic region, because there are no web angles to provide additional restraint once the seat has yielded. In addition, careful attention must be paid to the stability of the bottom angle and the beam web. As for Type I, the thickness of the seat angle is the controlling parameter.

Type IV: These consist of combining the simplest shear connection used in steel frames (bolted double web angles or single plate shear tabs) with the slab reinforcement. Although the angles or plate are relatively weak, the moment capacity of the composite connection can be substantially improved by increasing the thickness of the angle and lowering its position towards the bottom of the beam web. Since the web angles are carrying both shear and moment, care must be taken to prevent any type of block shear failure.

The main advantage of semi-

Figure 3.



Tests have been conducted on a variety of semi-rigid connections. Pictured above is a typical seat angle, a double angle shear connection to the web, and a continuous slab reinfromcement across column lines.

rigid connections is that they can be easily detailed to limit their strength, so that problems with local buckling of the beam flanges near the connections, shear yielding of the column panel zone, and formation of weak column-strong beam mechanisms can be avoided. The most efficient type of connection is one that behaves as a full restraint for service loads, yields at low drifts (0.5 percent to 0.75 percent) and has sufficient ductility and toughness to insure good hysteretic behavior up to 2 percent drift. Thus a semi-rigid, partial strength connection (Figure 1) is the most efficient solution.

The detailing provisions for both the slab steel and the connection elements are the key to good performance. The longitudinal slab steel should be kept with a column strip less than or equal to six column flange widths, and should extend at least 12" past the point of inflection. The bar size should be kept small (less than a #5), and at least three bars on either side of the column should be used. Transverse steel must be provided at each column line, and must extend at least 12" into the slab strip.

Serviceability Considerations

To reduce serviceability problems, a minimum of 0.1 sq. in. of steel per lineal ft. must be provided over the girders, with this reinforcement extending at least 24" on either side of the girder. Care must be taken that slip critical bolts are used everywhere, that local buckling of the beam flange or web in negative moment regions does not occur, and that yielding of the column panel zone be avoided. Full shear connection should be provided since the effect of partial connection has not been investigated.

Performance Studies

To study the performance of

Figure 4.

Consider a composite connection containing eight #4 bars on a fully composite W18x35 A36 beam with an effective width of 108". When transformed to an equivalent area of 36 ksi steel, the rebar steel transforms to $1.6(^{60}/_{36}) = 2.67$ sq. in. of steel. This is about the same area as that of the flange of the W18x35.

If a top and seat angle connection are selected, and an angle with about the same area as the top flange of the beam used, the M_{ult}, usable ultimate moment, would be about 700 kip-in. at a rotation θ_{ult} of about 24 milliradians, with an initial secant stiffness on the order of 110 kip-in./milliradian at 4 milliradians (θ_{ser}). Assuming A36 steel, an equivalent tensile stress of $^{700}/_{18(2.55)} = 15.2$ ksi or only about 40 percent ($^{15.2}/_{36}$) efficiency of the material.

In contrast, a semi-rigid composite connection will have a M_{ult} of about 2200 kip-in. at 24 milliradians, and an initial stiffness of 1200 kip-in./milliradian at 4 milliradians (θ_{ser}). This results in about a 70% efficiency of the material, and in tests more than a 90% efficiency has been achieved at rotations of about 50 milliradians. Thus, the composite semi-rigid connection has an initial stiffness of almost 20 times that of the steel-alone connections, and a usable ultimate strength three times higher than for comparable amounts of connection steel.

While the gains in stiffness and strength will not always be this large, this example clearly points out the potential of the system.

Figure 5.

Pictured above is a full-scale, two-bay, one-story subassemblage allowing the testing of a type of semi-rigid connection.

structures with Type I connections, 27 fixed-base, three-bay frames having four, six and eight stories, story heights of 12′, 14′ and 16′, and girder to column length ratios of 2:1, 2.25:1, and 2.5:1 were designed according to the LRFD Specification. Each frame configuration was analyzed with: (1) rigid connections; (2) semi-rigid connections; (3) semi-rigid composite connections and composite girders with varying moments of inertia. The frames were subjected to two load cases: (1) increasing both dead, live, and lateral loads proportionally until collapse occurred; and (2) increasing the lateral load only.

The composite frame design process was as follows: (1) design the frame as being rigid non-composite; (2) use the same column sizes; (3) replace the steel girders with steel girders capable of resisting the factored construction load case without yielding; (4) provide enough shear connectors to the selected steel girders to have 100% composite action; and (5) replace the rigid connections by semi-rigid composite connections. This design method proved itself to be reliable for the vast majority of the three bay frames that were studied. Only composite frames having stories of 16' with aspect ratios smaller than 1:1 were unserviceable.

The composite semi-rigid connections and frames described here are not too different from what is being built today. The main differences are in the slab and shear stud detailing and, most importantly, in the recognition in the analysis phase of the actual behavior of the connections.

Roberto Leon, Ph.D., is a professor in the Department of Civil and Mineral Engineering at the University of Minnesota in Minneapolis. ■

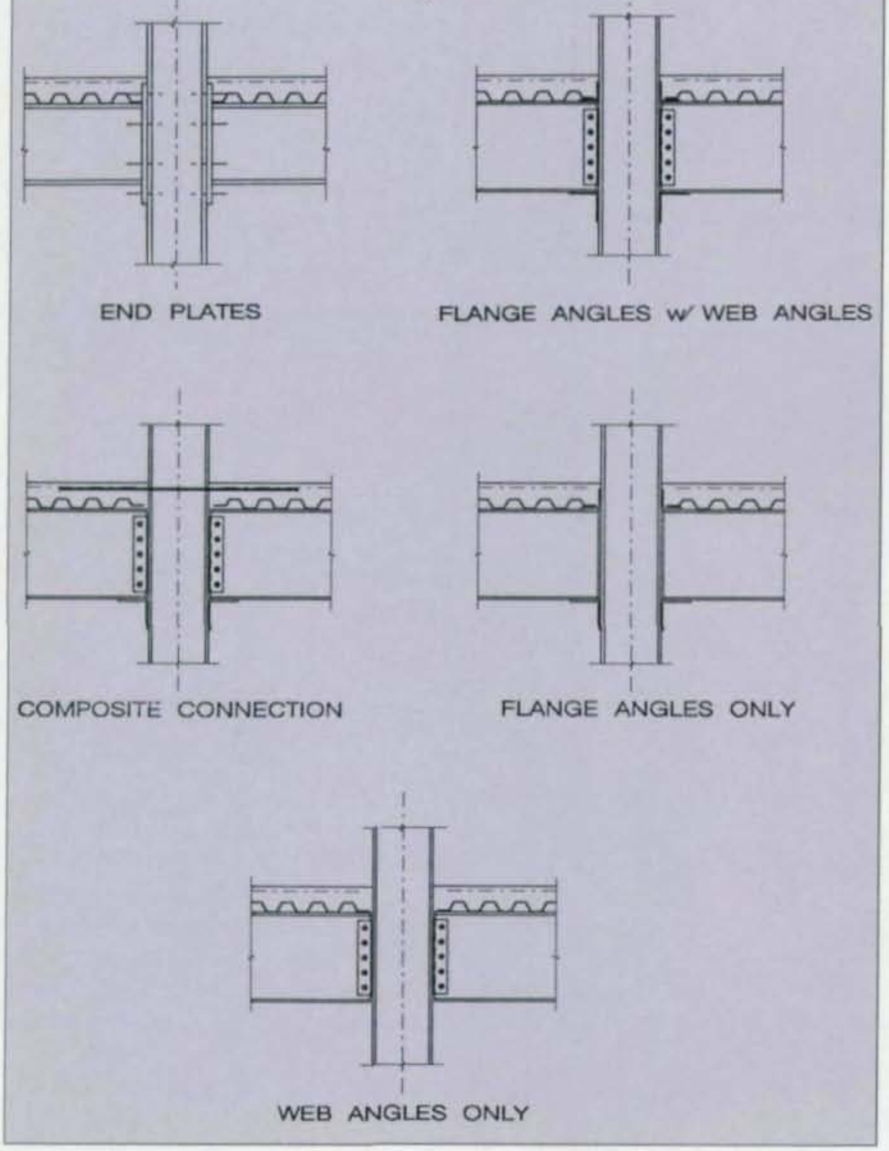
Figure 5a.

Pictured above is another view of the full-scale, two-bay, one-story subassemblage allowing the testing of a type of semi-rigid connection.

Unlocking The Inherent Stiffness Of Low-Rise Buildings

Industry standard shear connections provide restraint to columns and can be used as partial restraint connections in design

Figure 1.



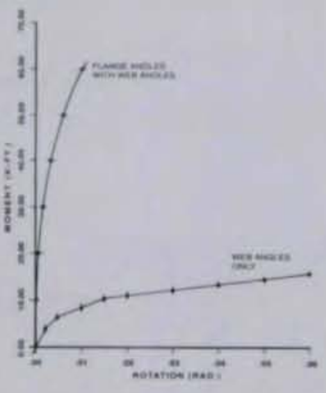
By Kurt Swenson, P.E., Ph.D.

Modern design specifications, such as the LRFD Specification, have created new opportunities for structural engineers to minimize steel frame costs. However, we can also learn lessons from the engineers of the past.

Before the development of advanced analysis and strict code requirements for lateral load design, most design engineers did not include lateral load analysis explicitly in their designs of low-rise buildings because they understood by experience that the inherent stiffness of the building frame was adequate to resist the small forces involved. Today, engineers tend to provide discrete lateral load resisting systems and ignore a large portion of the building in lateral load analysis. This separation of "gravity" and "lateral" systems results in a waste of material, labor and money.

By realizing that no connection is a pure pin and accounting for the inherent restraint in standard connections, the design engineer can "activate" the entire steel frame to resist lateral loads and produce a more efficient and economical steel frame for low-rise construction. Research has shown that typical shear connections provide restraint to columns and can be used as partial restraint (PR)

Figure 2.



connections in design.

PR connections represent an area in which the structural designer may reduce the cost of a structure and decrease the construction time for the steel frame. The cost reduction is realized in several areas:

- elimination of field welded connections;
- elimination of column web stiffeners;
- reduction in foundation costs;
- reduction of frame erection schedule;
- elimination of field welding inspection;
- simplification of beam-column connections.

Further economy can be gained by accounting for restraint provided by PR connections in the design of floor systems. However, this article will concentrate on PR connections in lateral load resisting systems.

Successful Applications

Even with the challenges presented to the design engineer, it has been the experience of this engineering firm that the extra work required pays off in reduced building costs which keeps clients happy. One particular application which holds great potential is in the low-rise office market.

In most areas of the country where design wind speeds are in the 70 to 85 mph range, low rise office buildings (two to four floors) with a regular grid and a medium to large floor plan (20,000 to 40,000 sq. ft.) can be constructed without special lateral load resisting elements. By utilizing all columns in the building to resist lateral loads through PR connections, a designer can eliminate the need for any Full Restraint (FR or "Fixed") connected moment resistant frames with their requisite stiffeners and full penetration welding.

In our experience, the required increase in connection stiffness is small and can be accounted for with slightly thicker double angles and two to four additional bolts per beam-column connection. The resulting design will increase the size of columns due to the increase in K and the additional moments imparted to the columns.

By spreading the lateral overturning moments throughout the structure, there is a reduction in the total foundation costs as the whole weight of the building is used to resist the moments at the foundation level. Also, the fabrication and erection costs of the steel frame are reduced by the elimination of field welding and stiffeners. Because the lateral loads are distributed, deep beam elements on frame lines are eliminated, which could reduce the floor thickness and the building height, resulting in further cost savings. Finally, the elimination of field welding will reduce the inspection costs, the chance for errors, and the time required to erect the steel frame, which is vital in a competitive commercial office market.

Another area where our office has been successful in the application of PR connections is in the evaluation of older buildings. Many older buildings do not have the discrete lateral load resisting frames used today, but many have top and bottom flange angle connections that provide significant stiffness at the connection. If the inherent restraint of these connections can be accounted for in lateral load analysis, the designer may eliminate or reduce the need for additional bracing or other more intrusive strengthening elements.

Optimizing Stiffness

The behavior of structures utilizing PR connections under lateral loads is greatly affected by the connection restraint available in the connections.

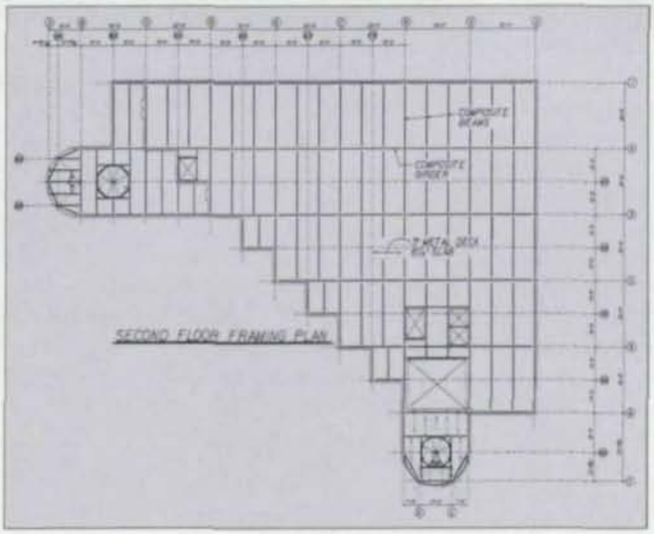
Figure 1 shows various types of PR connections. The moment rotation curves for two types of PR connections are shown in Figure 2. The difference in the amount of continuity or restraint provided by the two connections is readily apparent. The structure using relatively stiff connections with both clip and web angle connections will drift less, have more end moments in the girders and lower K factors than a structure using relatively flexible web angle or standard shear connections.

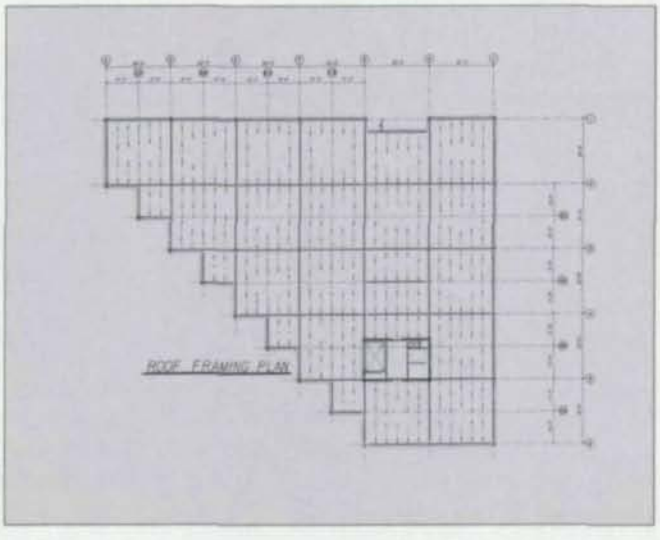
This variation in behavior creates an opportunity for the designer to optimize the structure's resistance to lateral loads by providing only the restraint required. The designer can essentially "dial-a-stiffness" for the structure by varying the connection stiffness once the gravity load design is satisfied. For structures with longer clear spans, the designer may want to take advantage of the inherent stiffness of the floor frame with stiffer connections while a structure with light floor framing and more columns may call for connections providing less restraint.

Second Order Elastic Analysis With Factored Loads Required

Because PR connections are nonlinear by nature, frames with PR connections must be analyzed under ultimate loading conditions to insure the stability of the structure. LRFD provides for this type of analysis. To predict the behavior of the structure at its true limit state, the analysis must include the non-linear moment rotation behavior for the connections specified through the range of possible factored loadings. Thus, the most practical method of analysis is to use a second order elastic analysis with the PR connections modeled as rotational springs with a known rotation curve.

The General Accident Insurance (Nashville Branch Office) building was designed by structural engineer Stanley D. Lindsey and Associates using PR connections. Architect on the project was Robert Lamb Hart, Planners and Architects.
Figures 3 and 4 *show the simplified floor and roof framing for this project.*





The moment magnifiers B1 and B2 contained in Chapter H of the LRFD Specification cannot model the reduction in stiffness that occurs as a PR connection approaches design loadings. The expressions for B1 and B2 were developed using connections with a constant stiffness, i.e. fixed or pinned, and should not be used when determining stability and design moments for lateral load resisting frames containing PR connections.

Since K factors are dependent on the restraint provided by the end connections and this restraint varies in PR connections, the second order elastic analysis must include a calculation of K for each load case based on the actual restraint provided by the connection at that loading. It is interesting to note that the K factor will vary not only with the lateral loads, but also with the amount of gravity load on the beams because the connection restraint is affected by the amount of moment at the end of the beams. In general, the K factors for columns will increase significantly due to the reduction in restraint at the beam-to-column joint.

A Design Example

The simplified floor and roof

framing for low-rise office building projects is shown in Figures 3 and 4 respectively. This project is located in Nashville and is subject to a design wind speed of 70 mph. The project contains approximately 40,000 sq. ft. on two levels with 30′ x 30′ typical bays. The floor system is a composite slab system with cellular deck designed to support a live load of 125 psf. The roof system consists of 1½" Type B metal roof deck, rigid insulation and a single-ply ballasted roof. The foundation system consists of shallow spread footings on a shot rock fill base.

Once the structural system was determined, the design of the structure developed in the following fashion:

1. Designed floor slab and beams for gravity loads;

2. Estimated column sizes for gravity loads with a K factor of 2.0;

3. Analyzed resulting frame with PR connections;

4. Increased connection stiffness as required to obtain desired drift and stability;

5. Checked member sizes and repeated steps 3 through 5 as required.

The resulting beam-column connections were double angle connections with no seat angles shown in Figure 5. The final double angle connections also are shown in Table 1 along with the connections which would be required to resist the factored shear force only. Comparison of the connections indicates that the increase in the connection material was small. Further, this increase occurred only at column connections, which reduced their impact on the total cost of the project.

Tables 2 through 4 show other properties of the structure determined during the structural analysis. Table 2 shows how the floor beam and girder moments varied from gravity to lateral load cases. The end moments are relatively small due to the somewhat flexible connections, and the lateral load case did not control the design. The comparison of frame drift in Table 3 indicates the large amount of drift that can be expected with PR connected frames when compared to FR connected frames.

Table 1. Comparison Of Simple Shear To PR Connections

		Angle Size	L	n
Floor	Shear Only			
	Girders	4"x3½"x¼"	16"	5
	Beams	4"x3½"x¼"	10"	3
Floor	PR Connection			
	Girders/Beams	4"x3½"x⅜"	15"	5
Roof	Shear Only			
	Girders	4"x3½"x¼"	10"	3
	Beams	4"x3½"x¼"	7"	2
Roof	PR Connection			
	Girders/Beams	4"x3½"x⅜"	12"	4

All bolts ¾" diameter A325 Bolts
Angles A36 material

Table 2. Factored Floor Beam Moments

	Negative Moments (K-Ft.)		Positive Moments (K-Ft.)	
	Gravity Only	W/Lateral	Gravity Only	W/Lateral
Beams	39	38	284	224
Girders	41	40	782	553

Table 3. Comparison Of Frame Drift At Service Loads

	Total Lateral Deflection (In.)	
	FR Connected Frame	PR Connected Frame
2nd Level	0.16	0.52
Roof	0.32	1.19

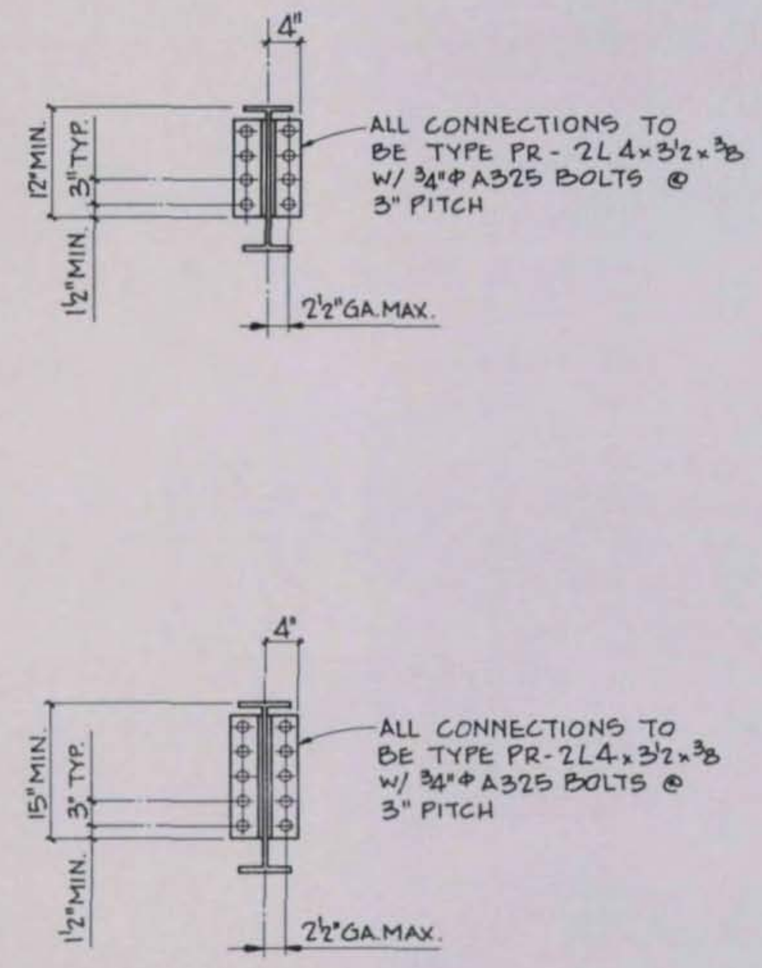
Table 4. Strong Axis Column Factored Moments And K-Values

Column Type	Gravity Only Moment (K-Ft.)	K	Gravity W/Lateral Moment (K-Ft.)	K
External	16	2.16	45	2.10
Internal	3	1.81	56	1.70

From this example, it is apparent that the real economy in using PR frames is realized when more frame lines can be utilized to resist wind. Table 4 presents the K factors and column moments used to design the columns. The typical first floor column size for this project is a W10x54 GR50. If sized only for gravity loads with a K=1.0, the typical interior column would be W10x39 GR50. This appears to be a significant increase in steel weight; however, if typical frames were utilized, approximately half of these columns would increase to W12x65 GR50.

By using very flexible PR connections in this building, enough connection restraint was mobilized to resist lateral loads within serviceability requirements. Thus, the

Figure 5.



lateral load resistance was provided with almost no increase in steel cost over a gravity load only design. In the final analysis, considerable savings were obtained and the steel frame cost almost 10 percent less using PR connections than with more traditional FR design.

Barriers To Use

In the design of a steel framed building, the inclusion of partial restraint connections may reduce the cost of the structure, but it also will increase the amount of work for the structural engineer. The PR connection is another variable that must be determined by analysis and then verified, designed, documented on the contract documents and inspected in the field. For this reason, many engineers do not take advantage of the economies of PR connections. It is much simpler to use a standard FR connection detail on the drawings; however, the increased cost may be the difference between a project being completed and one that is abandoned due to economic considerations.

Another barrier to the widespread use of partial restraint connections is the lack of access to moment rotation curves that are essential for the design process. To date, no industry wide organization has made the data on PR connection easily available to the design engineer. Without the available information, design engineers must take on the added burden of searching for available data from various sources. At this time, two of the best sources for a large database of moment-rotation curves for PR connections are available in the following publications:

- Kishi, N., and Chen, W.F., "Data Base of Steel Beam-to-Column Connections," Purdue University School of Engineering, 1986
- Goverdhan, A.V., "A Collection of Experimental Moment-Rotation Curves and Evaluation of Prediction Equations for Semi-Rigid Connections," Vanderbilt University, Masters Thesis, 1983.

These references contain valuable information concerning PR connection behavior, bibliographies for further reference, and experimental moment rotation curves for many different types of joints.

Use In Practice

When using a PR connected frame, the design engineer has to take extra steps to insure that the assumptions made in the design are realized in the field. This begins with the contract documents.

Each type of connection must be individually detailed, including angle sizes, thicknesses, and lengths, and it also is important to define the number of bolts, as well as the diameter, gauge, and pitch. Figure 5 provides an example of the connection details used on the project used in the example design. The location of each type of connection should be clearly marked on the plans and/or frame elevations. Finally, the engineer should include notes in the construction drawings as well as the specifications indicating the connections are not subject to redesign by the fabricator.

One main concern over the use of PR connected frames is that the in-place behavior of the connections will not match that shown in the laboratory due to construction tolerances and poor inspections. This is a valid concern. Improper fit-up, welding angles when bolt holes don't match, and insufficient tightening of bolts will greatly affect the connection behavior and cannot be allowed. Therefore, once the construction team is assembled, the engineer must stress the importance of the integrity of the connections, and must be involved in the construction review to insure that connections are correctly constructed.

Conclusions

In today's competitive construction environment where many projects do not reach groundbreaking because of budget constraints, an engineer must choose to use all of the available tools to reduce the steel frame cost of a building. The PR connection is a tool that allows the engineer to provide only the restraint needed by the frame, at a lower cost than regular FR connections. Many years ago engineers did not worry about lateral loads in the design of low-rise buildings because they knew, through years of experience, that the inherent stiffness of the frames were adequate. Now, our advanced analyses have begun to quantify this behavior so we can take advantage of it is modern steel design.

PR connections can be advantageous in some applications such as low rise buildings in low to moderate wind zones and during evaluation of existing buildings. However, the designer must respect the analysis required for a safe and serviceable design. The stability and drift of the frame must be verified under ultimate load conditions to insure an acceptable design. In addition, the designer must make special efforts to document the design and verify that it has been correctly executed in the field.

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Innovative Structure For An Award-Winning Airline Paint Facility

Though more attention has focused on this project's innovative mechanical system, it's structural system also is noteworthy

By Fred R. Schoenfeld, P.E.

When Delta Airline's Aircraft Strip and Paint Hangar in Atlanta received an ACEC Honor Award for Engineering Excellence, the jury focused on its innovative mechanical system which can transfer over 950,000 cu. ft./minute of air through the 617,000 sq. ft. facility.

But while the advanced stripping/painting technology and complex mechanical systems of the hangar are well deserving of attention, the structural design of the facility also is due its share.

Mechanical Requirements

Eight different design concepts were analyzed by Rosser Fabrap's architectural/engineering team and Delta, with the primary considerations being dictated by new environmental regulations, concern for employee health, and a desire for improved quality in painting and stripping operations. The final design divides the facility into 167,000 sq. ft. of aircraft bays arranged in an "L" shape and 450,000 sq. ft. of contiguous aircraft shops, specialty shops and storage areas arranged around the hangars on four floors.

Giant air handling units in the attic space of the hangar bays introduce fresh air from overhead into the bays. Air is returned near the floor at the side walls through paint arrestor dry type filters. Variable speed fan motors enable the

The sloping roof of the Delta Airline Aircraft Strip and Paint Hangar in Atlanta is supported on a series of large trusses, some clear spanning as much as 180'. In addition to the metal roof supported on top of the trusses, the bottom chords support a metal panel ceiling, which separates the conditioned space around the planes from the interstitial space above. A rolling work platform that was used to facilitate the installation of the mechanical equipment, such as the 8' ductwork and airhandling unit shown in the top photo. The shop framing shown in the smaller photo features typical bays of 25' x 40'. Of special note are the eight exhaust stacks in a mechanical well near the center of the photo.

air flow to be adjusted for stripping (high volume) and for painting (low volume) in each of the three hangar bays. During the stripping operation, liquid stripper is applied to the aircraft body and the excess is captured in stainless steel collector troughs beneath the aircraft and disposed of as hazardous waste. Annual capacity for the $67.5 million facility exceeds 240 aircraft.

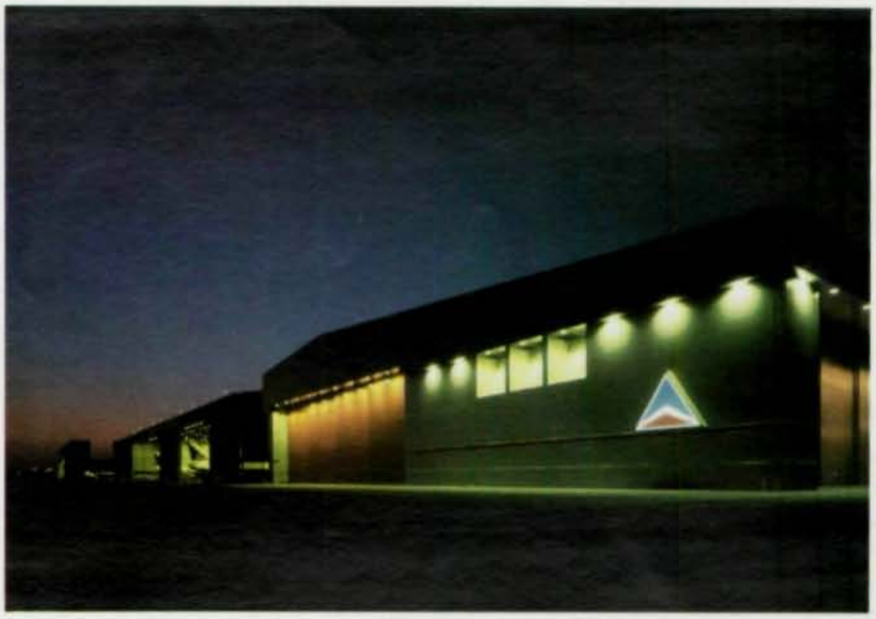
Structural Considerations

One of the design parameters that influenced the shape of the building and, as a result, the structural framing, was the sloping roof. Over the years, in its search for a leak-free, low-maintenance roofing system, Delta has settled on a site-rolled, continuous sheet, standing seam metal roof as its standard. But while the 1:12 slope of this roof reduces leaks and maintenance and provides in an attractive aesthetic, it also presents a structural challenge.

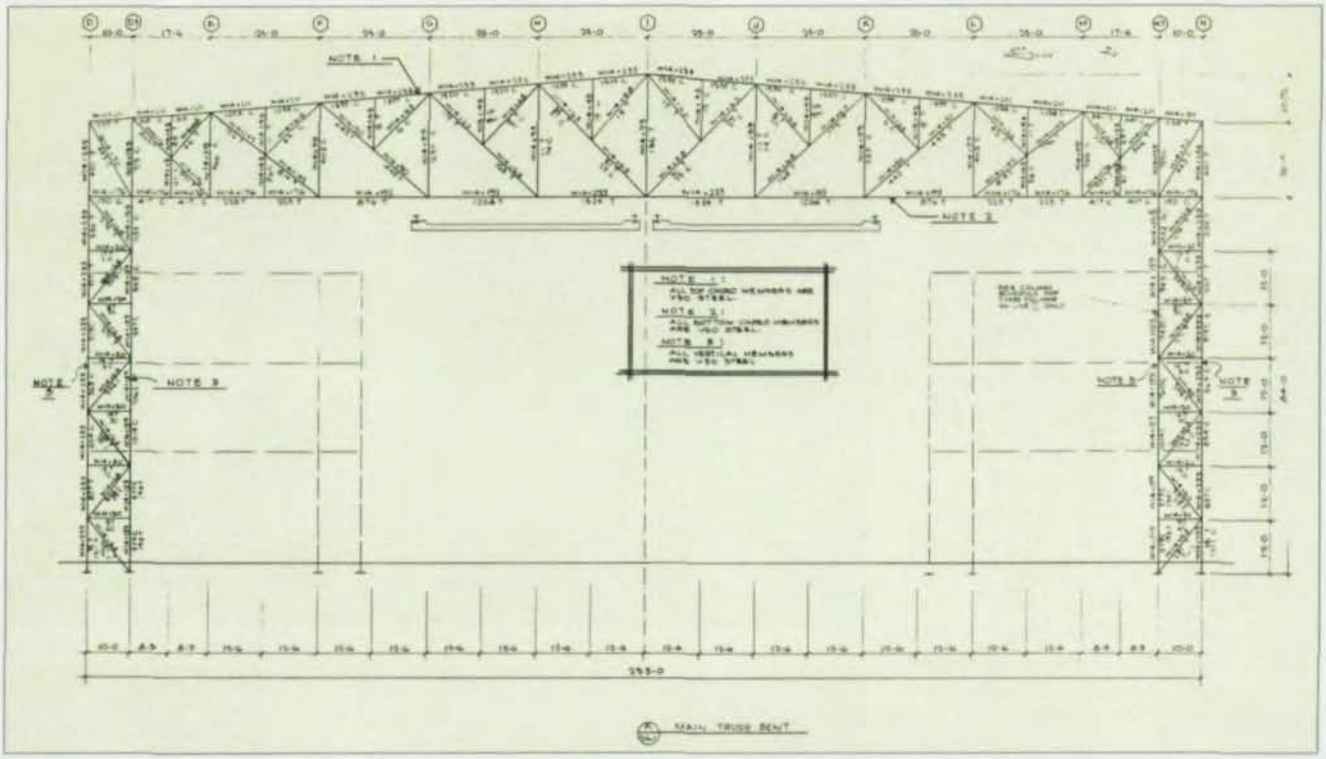
The L-shaped arrangement of the hangar bays, combined with the direction of roof slope and the fact that the main trusses need to span from side-to-side in the bays, resulted in two different roof framing systems.

The largest hangar bay (Bay 10) has six trusses with double pitched top chords spanning parallel to the slope of the roof. These trusses all have the same depth, occur at 54' on center and have a clear span of 235'. The truss depth goes from 16'-8" to 27'-3½". The two smaller bays (Bays 11 and 12) have five trusses with flat top chords spanning normal to the slope of the roof. These trusses vary in depth, occur at 50' on center, and have clear spans of 150' and 180' respectively.

The truss spacings of 50' and 54' were developed both for economics and necessity. A value engineering study developed with input from AISC-members Steel Inc., the project's fabricator, and Heaton Erecting, Inc., the project's erector, concluded that larger truss spacings were more cost effective. And these larger spacings also were necessitated by the need to



The aircraft bays are designed to accommodate the wide variety of aircraft in the Delta fleet. Top photo by Jim Roof.



accommodate the huge air handling units that are located in the interstitial spaces between the top and bottom chords of the trusses.

One feature that makes this hangar different from most others is a metal panel ceiling below the bottom chords of the trusses. This ceiling separates the conditioned space around the planes from the interstitial space above. The requirement of providing support for the ceiling, support for the air handling units and service catwalks, as well as bottom chord bracing for the trusses all without many complicated connections resulted in a unique three-layered framing system at the bottom chords.

The lowest layer consists of bar joists with ceiling extensions, which support the metal panel ceiling from their bottom chords. To keep the joists shallow, they span 27' in Bay 10 and 30' in Bays 11 and 12. The joists bear on angle seats suspended below the main truss bottom chords in Bay 10 and the sway truss bottom chords in Bays 11 and 12. Additionally, in Bay 10 the joists bear on the bottom flange of W27 girders, which occur midway between the main trusses.

The intermediate layer consists of the bottom chords of the trusses and the diagonal bottom chord bracing. Connections on the bracing are limited to the usual intersections with the bottom chords and, in the case of Bay 10, the W27 girders.

The upper layer consists of framing for the air handling units. By keeping spans short and the framing members shallow, this framing can pass above the bottom chord bracing. An exception to this is the W27 girders in Bay 10, which are a part of and contribute to all three layers. On the top layer, these girders support mechanical equipment framing flush with their top flanges.

Another feature of the hangar is the return air chases located immediately behind the metal panel walls of the hangar bays. These walls require girts and wind columns just like the exterior walls. In Bay 10, this lead to another interesting structural development. At main truss locations, two columns that occur on either side of a chase are tied together to form a vertical truss, which when joined to the horizontal truss that they support, create a trussed rigid bent. While the rigid frame action did not significantly help the horizontal span of the trusses, it did dramatically decrease lateral sway parallel to the trusses, which was very important in light of the lateral forces induced by the telescoping stacker cranes and the fact that Bay 10 was isolated from the rest of the structure by expansion joints.

The standing seam metal roof, while providing excellent protection against leaks in an unbroken field, does not work as well when it is penetrated. Therefore, to avoid penetrations, as well as giving the roof a clean, uncluttered look, all mechanical penetrations were limited to mechanical wells and vertical surfaces.

Shop Framing

While not as glamorous as the hangar bays, the shop framing deserves some mention. Floor-to-floor heights are 25' for the second floor and 20' for the third and fourth floors. The most typical bay size is 25'x40', with some 27'-6" x 40' and 30' x 40' bays. Design live load is 250 psf, but forklift traffic also had to be considered.

The slab consists of 5" of normal weight concrete on a 3" deep 18 gauge composite deck. Typical slab spans are 7'-6", 8'-4", and 9'-2". Slabs have continuous top and bottom reinforcement normal to the filler beams for crack control and to ensure continuing function even if loss of bond with the composite deck were to occur.

Filler beams, which span in the 40' direction, are designed as simple span composite beams, but sufficient reinforcing is provided in the top of the slab over the girders to handle full continuity under live load. Girders are designed as simple span composite members. A572 Grade 50 steel was used for both girders and filler beams after an economic study indicated it to be the most cost effective material. The project used a total of 8,500 tons of steel. General contractor on the project was Garrett-Patton Construction Co., Scottdale, GA.

Wind load design is based on an 80 mph basic wind velocity using provisions of ASCE 7. Except for the truss bents in Bay 10, lateral loads are resisted by X-type and chevron bracing, primarily at exterior walls. With so much gravity load leaning on the bracing, P-Delta effects were analyzed in the design of the bracing.

The building has a foundation of 16"-diameter augercast piles with 80 ton compressive capacity and reinforced for 20-ton uplift capacity at selected locations.

Tight Construction Schedule

As the result of a very aggressive completion schedule, the structural design of the hangar included consideration of ways to minimize construction delays. In order to facilitate field assembly of the hangar trusses and to keep field reaming of bolt holes to a minimum, truss connections were designed utilizing oversized holes, which meant the bolts had to be slip-critical.

All truss connections were made with 1"-diameter A490SC bolts using a Class A slip coefficienct. Truss members received a shop coat of primer, and faying surfaces in connections were masked. Proper bolt tension was ensured through the use of snap-off bolts.

Because the heavy mechanical equipment in the interstitial space over the hangers had to go in after the truss bottom chord framing was complete, but before the roof framing was finished, some careful sequencing was required. This became doubly critical when it was decided to use the same cranes to set the equipment as were being used to erect the steel.

To facilitate the mechanical contractor's installation of equipment and ductwork, a rolling work platform was created that rested on the top chords of the ceiling support joists and moved from bay-to-bay as the work progressed.

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Beneficial Procrastination

Delaying lead paint removal projects by upgrading the coating system can offer substantial benefits

By Eric S. Kline
and William D. Corbett

During the past 50 years, millions of gallons of lead-based paint have been applied to structures throughout the world. Recent regulations, however, on the containment and disposal of lead-based paint debris have resulted in rapidly escalating costs for repainting structures. In fact, these costs have increased so much that on some projects, the facility owner can no longer afford repainting, which results in delayed maintenance and potential steel section loss due to inadequate corrosion protection.

Because of the complexity and costs of lead paint removal and disposal, considerable efforts are being made to find alternatives to total removal of lead-based paint and the postponement of coating work. One alternative is to upgrade the existing system with one of several generic coating types requiring little surface preparation.

Choosing to "upgrade" the system may permit an owner to profit by "beneficial procrastination"—postponing the cost of coating removal without the possible consequences of corrosion damage affecting the integrity of the structure. This delay is designed to permit new technology to develop, which in turn should lower the costs associated with lead paint removal projects in the future.

Approaches To Upgrading

There are several techniques to achieve cost savings by system upgrading. These include: upgrading the coating system in more severely corrosive environments to reduce deterioration and to extend the service life; inspecting and repairing the coating system before extensive surface preparation is necessary; and partially removing the coating system in areas where total system removal is not required. For example, it may be as necessary to totally remove a peeling topcoat as it is to spot clean and prime isolated rusted areas.

A close-up of riveted structural steel on the Swinburne Bridge in Pittsburgh after "upgrading".

The first approach, applying a better paint system on severely corroded areas, is not a new concept. The SSPC developed the environmental zone painting system in the early 1970s. Therefore, this article offers guidelines for the second and third approaches: inspecting and repairing a system before extensive surface preparation steps are necessary; and partial removal where total system replacement is not necessary.

Can The Existing System Be Upgraded?

Three conditions of the existing system are analyzed to determine if it can be upgraded. These include the extent of the corrosion on the structure, the total thickness of the existing coating system (including the number of coating layers), and the adhesion characteristics of the system.

To determine if a coating system can be upgraded, the coatings or corrosion engineer must first conduct a visual assessment of the amount of corrosion on the structure. SSPC-Vis 2, "Standard method of Evaluating Degree of Rusting on Painted Steel Surfaces" (also referenced as ASTM D-610), is a method commonly used to establish the amount of corrosion.

SSPC-Vis 2 uses photographic reference standards to illustrate the degree of rust on a numerical scale from 0 to 10. Note that the rust grade scale is in reverse order with a higher rating indicating less rust. Rust Grade 10 indicates no rust or rust covering less than 0.01 percent of the surface while Rust Grade 0 corresponds to 100 percent of the surface rusted. However, because employing SSPC-Vis 2 on complex structures can be difficult, photographic standards may need to be custom-made for bridges and other structures.

Regardless of the method employed to analyze the data, areas with 10 percent or more of the surface rusted would generally not be

Physical testing often examines substrate condition, total coating thickness, adhesion, and damaged coating thickness.

candidates for upgrading by topcoating, but rather would need extensive degrees of surface preparation, perhaps including total removal. Areas with less than three percent of the surface rusted, however, would be prime candidates for upgrading if the existing system is able to withstand the rigors of some minimal surface preparation and the contractive curing stresses and additional weight of the newly applied coating system, all of which can be determined with physical testing.

Physical Testing

Depending on the age of the structure, many different coating layers may be present, and these may vary both in thickness and generic type. The cohesive strength of the coating layers, as well as the adhesion of the coating film to the substrate, must be assessed before adding new layers and curing stresses to the existing film.

The total thickness of the existing system can be determined using a non-destructive dry film thickness gage (magnetic pull-off or magnetic flux). The Tooke Gage, a destructive film thickness gage, can be used to determine the number of layers and the thickness of each.

Perhaps one of the most critical evaluations in determining whether or not the existing system will support an additional layer is its adhesion. Coating adhesion is evaluated in accordance with ASTM D-3359 "Measuring Adhesion by Tape Test."

The Decision Process

So when can a coating system be upgraded?

Typically, if the coating system exhibits less than 10 percent deterioration/corrosion, exhibits thicknesses in the 5- to 20-mil (125- to 600-micron) range, and has satisfactory adhesion, the engineer can be reasonably confident that the system is a candidate for applying an upgrade. However, even under these conditions test patches of the candidate system(s) should be applied before a commitment to upgrading is made on a wholesale basis on a large structure.

Alternatively, if the topcoat is peeling but layers beneath spot rust are intact, hand (SSPC-SP2) or power (SSPC-SP3) tool cleaning may be used to prepare surfaces before upgrading.

If the coating is very thick (25-40 mils or more) and the system's adhesion is poor to marginal, any decision to attempt recoating will necessitate the application of test patches.

Coating Selection

The selection of upgrade coating systems should focus on materials that have low shrinkage characteristics during curing and high solids content to minimize solvent penetration and softening of the underlying system.

Laboratory and six years of field testing have now been completed on the use of various high performance upgrading systems over back-to-back riveted angles removed from the steel bridge railing of the Ewing Park Bridge in Ellwood City (Pittsburgh), PA.

These angles contained a thick, lead-based alkyd coating (25-30 mils) that was brittle but otherwise mostly adherent. The angles were prepared using three different degrees of surface preparation: SSPC-SP10 (near white blast cleaning); SSPC-SP7 (brush-off blast cleaning); and SSPC-SP2 (hand tool cleaning) for the removal of obviously loose coating followed by blow-down using only compressed air.

These angles were then coated with five different coating systems:

- vinyl zinc-rich/high-build vinyl (7.5 mils);
- urethane zinc-rich/urethane topcoat (5 to 8 mils);
- zinc-aluminum-pigmented epoxy mastic/vinyl topcoat (7 to 9 mils);
- two-coat epoxy mastic (10 to 16 mils);
- two coats of an aluminum-epoxy-urethane-mastic (10 mils).

The laboratory exposure consisted of approximately 500 hours of accelerated weathering. Field exposure consisted of placing duplicate angles for each of the surface preparation/coating system variable beneath a leaking expansion dam on the Carson Street Bridge ramp in Pittsburgh. The panels were then examined by the authors visually in accordance with SSPC-Vis 2. Although the initial condition of the old alkyd presents an uncontrolled variable for panels cleaned according to SP7 and SP2 with blow down, the data suggest the following:

- All five systems performed well over surfaces prepared in accordance with SSPC-SP10. Slight evidence of underfilm corrosion (less than 1 percent rusted), perhaps due to the presence of residual chlorides from the initial field exposure, was noted on some systems with surfaces coated with the thin film systems (less than 10 mils total thickness). All of the areas cleaned in accordance with SSPC-SP10 performed better than those cleaned to SSPC-SP7 or SSPC-SP2 with blowdown.

- It was apparent that areas prepared using only air blow-down performed as well or better than areas that had been Brush-Off blast-cleaned. This result was unexpected, and may be attributable to a fracturing and weakening of the old alkyd due to the impact of the abrasive.
- On the blow-down and Brush-Off blast-cleaned areas, the thicker film systems performed better than the thin film systems. When the epoxy mastic or urethane mastic was used, minor defects were noted on the blow-down and Brush-Off blast-cleaned surfaces; however, the overall performance approached that of the SSPC-SP10 surface.

Other Case Studies

Additional evidence of the viability of system upgrading is found in data that a major chemical company has developed from two major bridges across the Monongahela River in the Pittsburgh area. Surface preparation involved spot cleaning of rusted areas in accordance with SSPC-SP6, followed by a complete brush-off blast in accordance with SSPC-SP7. In each of these cases, the vast majority of the old alkyd was allowed to remain on the steel surface. The coating system used on these two structures consisted of an aluminum-filled, moisture-cured urethane spot primer; a full intermediate coat of aluminum-filled, moisture-cured urethane; and a polyester-aliphatic-polyurethane topcoat.

After 13 years exposure, both bridges are in good condition and exhibit little corrosion and virtually no signs of coating disbondment with the old alkyd system.

On other projects, a 100 percent solids penetrating epoxy sealer has been used. One project, the Swinburne Bridge in the Pittsburgh area, was coated using this system in 1989. The existing coating system was a lead-based paint, greater than 30 mils thick, exhibiting cracking and marginal adhesion. The existing system was Brush-Off Blast-Cleaned (SSPC-SP7) to removed loose, old paint. After Brush-Off Blast Cleaning, hundreds of thousands of coating islands remained. In addition to the 100 percent solids penetrating epoxy primer that was applied, an epoxy intermediate and urethane topcoat also were applied. After three years, the coating remains intact and adherent, and no spontaneous disbonding within the old coating layers is occurring, even in areas abraded by the impact of gravel or stones.

An angle from the steel bridge railing of the Ewing Park Bridge in Ellwood City, PA, in "as received" condition.

Angle from the steel bridge railing after surface preparation. Left to right: SP7, SP2/blow down; SP10; SP2/blowdown; SP7.

Angle from steel bridge railing after applicatoin of candidate upgrade system.

Conclusions

The overall advantage of upgrading the existing system is reduced maintenance costs now; with the hope that the coating system life will be extended. In addition, our study suggests that simply coating over the old system after removing obviously loose paint by hand tool methods (SSPC-SP2) may be an effective approach.

Total lead paint removal may cost as much as $5 to $10 per sq. ft. It becomes apparent that minimal surface preparation, followed by upgrading with one or more of the materials discussed, would provide substantial cost savings by reducing coating costs to approximately $2 per sq. ft.

However, it must be recognized that each project is unique. The physical characteristics of the existing system must be carefully evaluated to determine whether it is a candidate for upgrading.

This article is condensed from an article appearing in the March 1992 issue of the Journal of Protective Coatings & Linings and was reprinted with permission. Eric Kline is a senior coatings consultant with KTA-Tator, Inc., and is the SSPC representative to the Council for the Advancement of Steel Bridge Technology. William Corbett is marketing manager at KTA-Tator. ■

New Coating Systems For Bridges

The pending obsolescence of higher volatile organic content (VOC) coatings has resulted in the rapid expansion of the protective coating market in the 1980s and the introduction of many new state-of-the-art formulations

By Tom Calzone

Most everyone involved with new bridge construction coating projects has seen changes in the nature of products they apply and specifications for coating work. Generally, polymer technology used for VOC compliant coatings formulations has resulted in improved performance. And along with this you get different application properties that effect the way steel is painted in the shop.

But in addition to the changes in formulation, the variety of products and systems has increased several fold since 1980. (Currently specified, new construction, shop-applied bridge çoating systems are described in Table 1.) Multi-coat shop systems, cleanliness requirements, materials costs and inspection scrutiny have dramatically increased the cost of shop coating. Now more than ever, fabricators must get a handle on their coating related costs.

VOC compliant formulations of the common generic types are accomplished in two ways. Via water based technology or by increasing the solids content. In most cases both approaches require formulation of entirely new products to replace noncompliant technology.

Table 1:

System Specified For Shop Application	VOC Compliance	Note
Inorganic zinc, solvent based	Compliant and noncompliant	Predominant system
Inorganic zinc, water based	Compliant	Zero VOC content
Inorganic zinc/high build urethane	Compliant and noncompliant	
Inorganic zinc/ epoxy/urethane	Compliant and noncompliant	
Inorganic zinc/vinyl	Noncompliant	Vinyls cannot comply
Epoxy zinc rich/ epoxy/urethane	Compliant and noncompliant	
Epoxy mastic/epoxy mastic/urethane	Complaint	
Alkyds - 2 or more coats	Compliant	
Acrylic latex - 3 or 4 coats	Compliant	Mostly used as zinc topcoats
Epoxy zinc rich - 2 coats	Noncompliant	One state

Inorganic zinc remains the predominate shop construction primer. The reasons for its growing popularity as a shop primer through the 1970s make it the product of choice today even with the advent of many new options. Among the benefits of inorganic zinc:

- Unsurpassed corrosion resistance.
- Inorganic zinc primed steel can be handled quickly with minimal damage in the shop, in transporting and erecting.
- Undercutting corrosion from damage points, abraded edges or holidays is virtually nonexistent. Inorganic zincs are considered to be permanent primers and expected to last for the design life of the structure. Remedial maintenance of topcoats may be necessary over that period but should not entail blast removal.
- Traditional formulations can weather for years in the yard or at the erection site with no special preparation prior to topcoating.
- Inorganic zincs are compatible with a variety of finish coats.
- They achieve Class B friction ratings according to the "Specification for Structural Joints using ASTM A325 or A490 bolts." This high friction value remains during long weathering periods prior to erection and fastening.

The traditional inorganic zinc

formulations do not comply with solvent emissions (VOC) regulations in many states and therefore fabricators are applying VOC compliant inorganic zincs. However, compliant formulations cost more than their noncompliant predecessors.

There are two methods to formulate VOC compliant inorganic zincs: high solids-solvent based and water-based. Both approaches result in inorganic silicate binders holding zinc particles on the steel. Each formulation will behave differently in application than the classic solvent based zinc.

High Solids Solvent Based Inorganic Zinc

Naturally, each manufacturers' high solids formulation is different. There are some commonalities, however, that affect application. A painter using traditional techniques will tend to build extra coating thickness with the high solids products. Resistance to mudcracking (a function of thickness) varies between manufacturers, however, with all products thicker films mean more consumption. For the most part the high solids, solvent based materials will behave in familiar fashion.

Water Based Inorganic Zinc

While water-based inorganic zincs are not new, they had seen little if any use as bridge primers prior to the restrictions imposed by VOC legislation.

Water based inorganic zincs have significantly different application requirements and curing properties. While the solvent based inorganic zincs require atmospheric moisture to cure, the water based products rely on evaporation of moisture from the coating to proceed with the curing reaction. Water based inorganic zincs contain no co-solvents and will dry at the rate of water evaporation. This rate may vary greatly with temperature and relative humidity. The solvent-borne materials contain co-solvents of varying evaporation rates and may be thinned with fast or slow solvents as needed. This provides for a more consistent application throughout a wide climate range. These differences are readily apparent to the painter.

The fabricator must approach surface preparation and cleanliness differently with water-based inorganic zincs. These primers require a jagged profile as produced by a steel shot/grit blast mixture. This presents a lot of surface area for adhesion. Reduced adhesion will occur on flame hardened edges due to their resistance to blast profiling.

Some fabricators will perform an additional blast on these surfaces to promote adhesion. Also, traces of oil contamination will prevent adhesion. Oil and water don't mix. A trace of oil which would have no effect on solvent based products prevents water borne inorganic zinc from effectively wetting out the steel and adhering. Solvent wiping blasted steel will only spread the affected area. Steel must be thoroughly degreased *prior* to entering the blast cabinet so that it comes out with no traces of contamination and does not contaminate the abrasive. After blasting, the steel must be primed immediately or protected from contamination from adjacent painting and other sources of organic contamination (Salamander heaters, hand prints, etc.).

Table 2 shows the similarities and differences between traditional inorganic zincs and low VOC formulations.

The low VOC formulations of other shop coatings types also differ from their predecessors. These

Table 2: Traditional Inorganic Zinc Vs. Low VOC Formulations

	Water-Based	Solvent-Based
VOC Content	0	2.2 - 3.5 #/gal
Performance	Excellent	Excellent
Mudcracking resistance	Less than traditional products	Varies by formulation
Surface preparation	Ultra-clean blast required	Same as traditional zincs
Application		
—Equipment	Conventional spray preferred	Conventional or airless spray
—Temperature	35 - 120 degree F	0 - 120 degree F
—Humidity	0 - 90% (see note 1)	30 95% (see note 2)
Topcoat time	As low as one hour	Same as traditional zincs
Hardness	Harder	Same as traditional zincs
Topcoat compatibility	Same as traditional zincs	Same as traditional zincs
Friction	Class B	Class B
Salting	Salt Removal may be required to topcoat	Same as traditional zincs

Note 1: At low humidities, dry spray tendnecy increases dramatically. High humidity may retard ure, effect adhesion and early rain resistance.
Note 2: Low humidity applications will lengthen cure time.

differences are summarized.

Epoxy Primers And Intermediate/finish Coats

New resin technologies have enabled manufacturers to comply with VOC regulations while improving chemical resistance, corrosion resistance and adhesion. The cost of new products (on a square foot basis) is the same or slightly higher than old technology.

The newer, high solids formulations are made with lower molecular weight resins more easily reduced to a sprayable consistency. However, pot life is reduced and drying is slowed. Fortunately, recoating of low VOC epoxies may occur earlier, since in some products a "wet-on-wet" second coat is acceptable due to a reduced tendency to trap solvent.

Again, with the newer coatings the painters natural habits will result in thicker films. Some adjustment will be necessary to realize economy from the solids content.

It is very important that the thinner used be matched to the solvents and resins in the product. The lower solvent content in the material leaves less room for marginally compatible thinners and following manufacturers recommended thinning practices is crucial.

Polyurethanes

Aliphatic polyurethanes are being specified more frequently for shop applications. The primary purpose of the urethanes is gloss and color retention. The performance of various VOC compliant formulations varies greatly and may be better or worse than earlier products used. Note also that there is a wide spread in cost between the VOC compliant products, and invariably they cost more than the old products.

High solids formulations are more difficult to apply in a car-like finish. Reduced solvent levels effect levelling and flow properties and can result in more of an "orange peal" appearance in the surface. This can be particularly evident in high gloss products. Again, as with other high solids types, painters must adjust for the improved coverage rates to keep material consumption down.

Acrylic Latexes

Several states are specifying acrylic latex products for their VOC new construction coatings, and several others are considering the move. House paint is typically an acrylic latex and considerable research has been done to improve weathering and color stability in latex resins for this huge market. Industrial formulations provide excellent adhesion, and corrosion protection as well.

The most common use of acrylic latex bridge coatings is for topcoating inorganic zinc, e.g. on Alabama, Florida and Virginia specifications. Acrylic latex formulations are compatible with a number of primers including epoxy zinc rich, epoxy mastics and even old lead bearing oil alkyd coatings.

Acrylics are significantly different from other coatings. The rheology, or appearance in the can, looks thick and puffy. The unfamiliar painter will be inclined to thin the material (with water). This may not be necessary because the thick appearing latexes are often sprayable as is, perhaps with some equipment modifications. A small amount of water can go a long way thinning latex products and too much can ruin the film-forming and curing properties.

Acrylic latex will become tough and have excellent adhesion. Unfortunately their adhesion and damage resistance develops more slowly than most solvent based products. High humidity will retard the cure by slowing water evaporation. Cold temperatures, 45°F being about the coldest allowable, do not substantially increase the dry to touch time but will greatly extend the full cure period.Acrylic latex has a lower tolerance of thick films. Applications of five or even four mils dry film thickness can cause the coatings to dry from the outside-in. The impermeable surface skin that forms will then crack as it shrinks with water and co-solvent trapped within the material. Special care is needed around fillets and stiffeners to avoid heavy build.

Summary

There are two approaches to VOC compliant coatings, via water based or high solids, solvent based technologies. The differences between familiar materials and new VOC compliant formulations are summarized:

High Solids Coatings

- Reduced working time from old formulations (pot life)
- Longer cure time (relative to pot life)
- Fast film build
- Less slickness in finish. Tendency to show "orange peal"
- Quicker recoatability, often "wet-on-wet"
- Thinning procedure is more critical

Water-Based Coatings

- Provide a long working time (pot life)
- Rheology differences may effect equipment used
- Prone to cracking at excess thickness
- Cure time in high humidity is extended

As always there are growing pains in assimilating new technology. These can be overcome with relatively small investments in equipment and training. Your coating supplier is trying to formulate user friendly products within the limits of the raw materials. Rely on your coating supplier to help with these new products. The manufacturer can reduce your routine headaches and costs and perhaps prevent a catastrophe from occurring. The supplier can also familiarize your inspector to the differences with the new technology, and as a team, put out a quality product of reasonable cost while complying with environmental mandates.

Tom Calzone is Highway Market Manager for Carboline. This article is adapted from a paper presented at the 1992 National Steel Construction Conference.

■

Corrosion Resistance

A high performance, single-component urethane coating from Wasser High-Tech Coatings eliminates most application restrictions for temperature, humidity and dew point. The company's coatings will cure as quickly as four hours in temperatures down to 15 degrees F, can be applied in humidities of 6 to 99 percent, and can be wetted or immersed after as little as 20 minutes after application. The coatings offer superior corrosion resistance and can be used to encapsulate red lead. They have been specified by more than a dozen state DOTs as well as more than 10 major utilities.

For more information, contact: Wasser High-Tech Coatings at (800) MC-PAYNT.

Cold Galvanizing

Z.R.C. Cold Galvanizing Compound stops rust and rust creepage on ferrous and nonferrous metal surfaces.

For more information, contact: ZRC Products Co., 21 Newport Ave., Quincy, MA 02171-9975 (617) 773-1180; fax (617) 328-5304.

Fast-Curing Epoxies

Tnemec Co. has introduced Series 160 Tneme-Fasprime and Series 161 Tneme-Fascure. These fast-curing, high-performance coatings can be handled in two hours and recoated in three, making them ideal for shop applications. Tneme-Fasprime is a rust-inhibitive primer for steel where extra corrosion resistance is needed.

For more information, contact: Tnemec Co., Inc., P.O. box 411749, Kansas City, MO 64141-1749 (816) 483-3400; fax (816) 483-1251.

Environmentally Conscious Coatings

Southern Coatings, a subsidiary of Pratt & Lambert, Inc., is a leader in providing environmentally conscious primers and topcoats for the steel fabricator and joist manufacturer. The Enviro-Guard line offers lead- and chromate-free primers and coatings offering superior protection against rust and corrosion.

Complete information on Enviro-Guard VOC compliant primers, as well as Chemtec 606 Water Base Epoxy Zinc Rich Primer, Chemtec 608 Inorganic Zinc Rich Primer and Dura-Pox 646 Epoxy Mastic High Build system, is available by contacting: Southern Coatings, Inc., P.O. Box 160, Sumter, SC 29151 (800) 766-7070; fax (803) 254-4833.

Rust Overcoating

PRE-PRIME 167 from Devoe Coatings is a rust penetrating sealer designed to wet, strengthen and seal porous rust. The product is a 100 percent solids, two-component epoxy. This thin coating wets and wicks its way through rust.

For more information, contact: Devoe Coatings Co., 4000 Dupont Circle, P.O. Box 7600, Louisville, KY 40207 (502) 897-9861.

Low VOCs

Available from Glidden and ICI Paints is Lifemaster Pro, a waterborne acrylic enamel with a low-level of VOCs. The coating is ideal for both structural steel and metal siding.

For more information, contact: Glidden Industrial Coatings, 801 Canterbury Road, Westlake, OH 44145 (216) 892-5341.

Coating Thickness Measure

A new range of Coating Thickness Gauges has been introduced by Elcometer Instruments Ltd. The Elcometer 345 is pocket-sized, yet offers a number of advanced features, including an angled screen with backlighting, so that even in the darkest corners these instruments can be easily read. Options include the entry of limits for tolerance checking and the ability to average readings. Also, one model offers memory for up to 10,000 readings.

For more information, contact: Elcometer Inc., 1893 Rochester Industrial Dr., Rochester Hills, MI 48309 (800) 521-0635; fax (313) 650-0500.

Steel Maintenance

Carbomastic 15 Low Odor from Carboline is recommended for the maintenance painting of rusty steel or for upgrading old coatings on steel bridges, metal buildings, and exposed structural steel. Only a single coat is required for most applications and hand or power tool cleaning often is acceptable. The product has excellent immersion resistance to both salt and fresh water and resists acid, alkali and solvent spillage.

Carbo Zinc 11 is used as a single coat protection of steel structures in weathering exposure and as a base coat for organic and inorganic topcoats in more severe services. The self-curing, inorganic base coat protects steel galvanically, eliminating sub-film corrosion.

For more information, contact: Carboline, Technical Service Department, 350 Hanley Industrial Circle Ct., St. Louis, MO 63144-1599 (314) 644-1000; fax (314) 644-6883.

Technical Information

The Technical Information Packet Service (TIP) from the Journal of Protective Coatings & Linings provides a collection of photocopied information previously published in the magazine. A typical TIP runs 50 to 100 pages. Subjects include: Achieving VOC Compliance; Cleaning and Painting Weathering Steel; Lead Paint Removal; Generic Coating Types and Their Uses; Coating Economics; and total Shop Painting. The base cost is \$5 + \$0.20 per page. Customized TIPS also are available.

For more information, contact: TIPS, (800) 837-8303.

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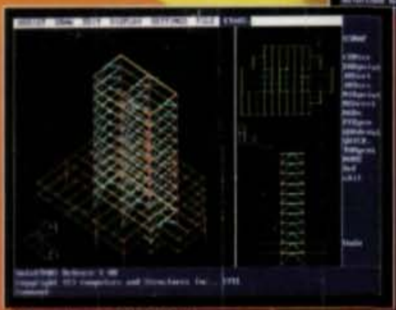
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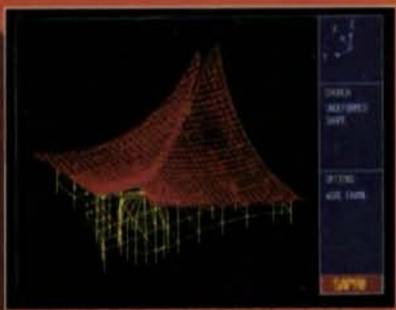
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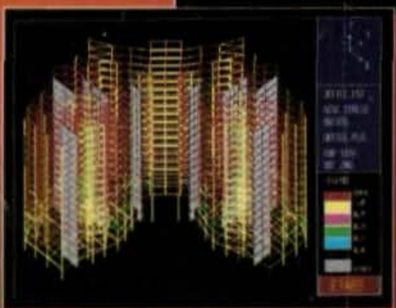
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