MODERN STEEL CONSTRUCTION

April 1995

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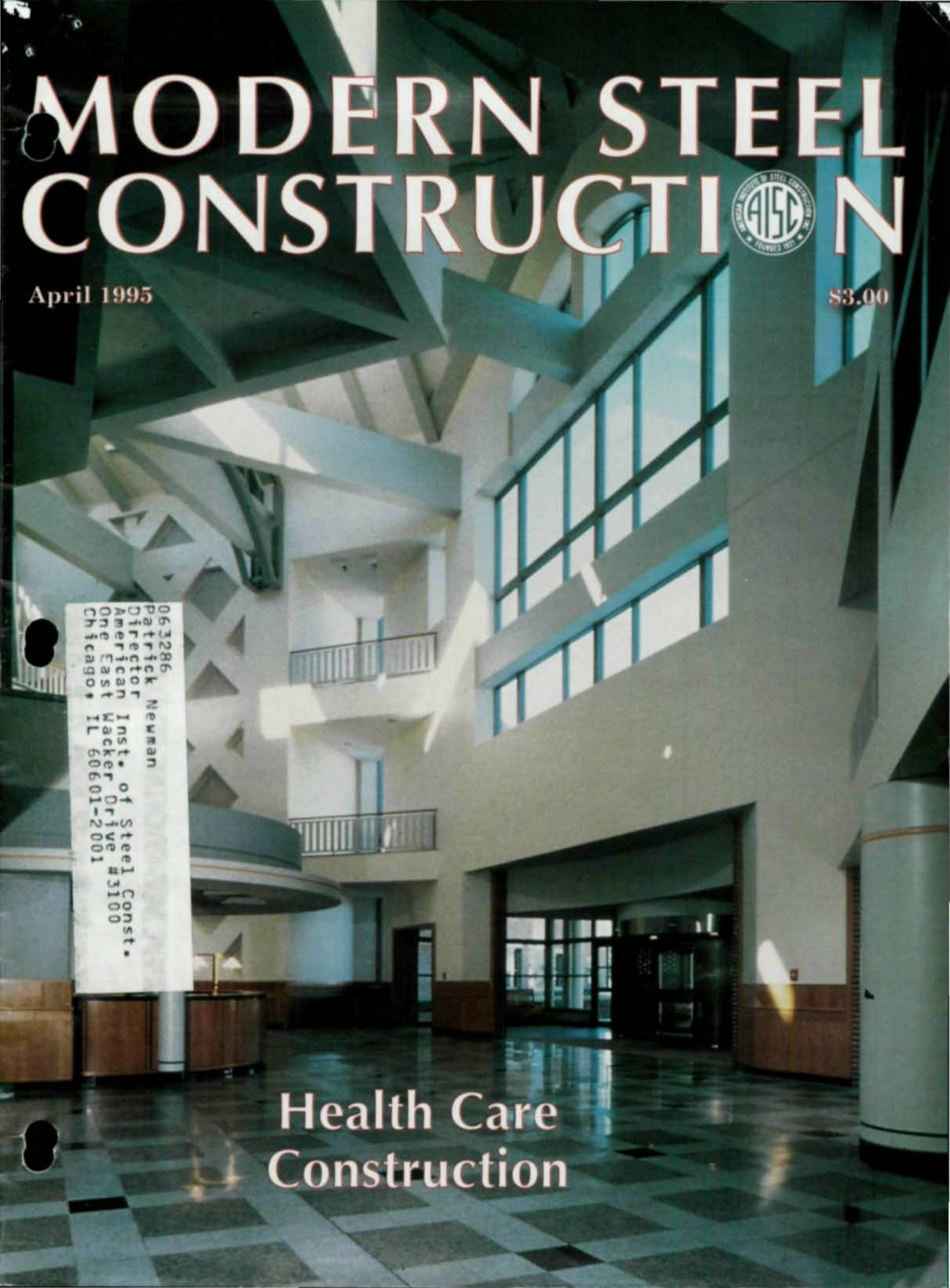


Health Care Construction

063286
Patrick Newman
Director
American Inst. of Steel Const.
One East Wacker Drive #3100
Chicago, IL 60601-2001

USD UNITED STEEL DECK, INC. deck design data sheet #22

Uplift Values for SCREWED DECK

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

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

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

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

Key

- Fu = 60 ksi
- Fu = 45 ksi
- Fu = 58 ksi

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

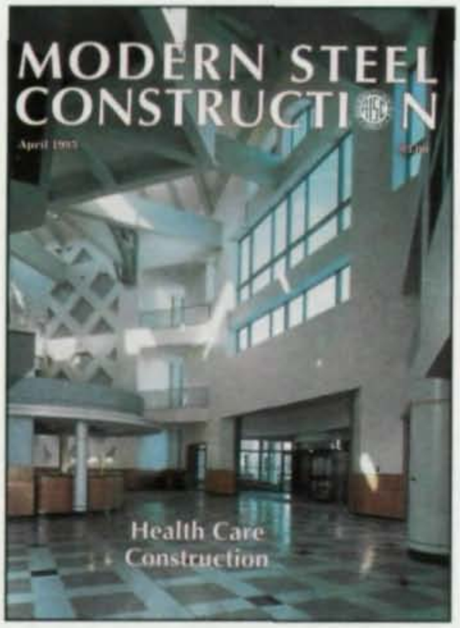
Screw	1/4"	3/16"	10 ga. (0.135)	1/8"	12 ga. (0.105)	14 ga. (0.075)	16 ga. (0.060)	18 ga. (0.047)	20 ga. (0.036)	22 ga (0.030)
#10	2.34	1.76	0.98	1.17	0.76	0.55	0.44	0.35	0.26	0.22
#12	2.66	2.00	1.12	1.33	0.87	0.62	0.50	0.38	0.29	0.24
1/4	3.08	2.31	1.29	1.54	1.00	0.72	0.57	0.45	0.34	0.28

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

Please circle # 33

MODERN STEEL CONSTRUCTION

Volume 35, Number 4 April 1995



A three-story, skylit interior "concourse" helps visitors and patients easily find their way around the Lancaster General's new Health Care Campus. The story behind this innovative design begins on page 24.

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QUALITY CERTIFICATION

THE DECEMBER ISSUE OF *MODERN STEEL CONSTRUCTION* MAGAZINE FOCUSED ON THE AISC QUALITY CERTIFICATION PROGRAM. The Certification Program article created some interesting comments, mostly praiseworthy, but some suggesting that just because a fabricator is not AISC certified does not mean the company does not produce good quality. Other comments reflected a perception that the AISC Quality Certification Program was designed for large plants. Perhaps a comment on why AISC feels so strongly about quality certification would be in order.

In the mid-1960s, quality became a buzz word in the United States. Through the works of Deming and others in Japan, that country emerged from a perception of low quality to a reality of high quality products. Deming, Crosby and others became the oracles for product quality and procedures to deliver that quality. Quality programs in many forms promoted under a variety of names became the subject of seminars, articles and books. Some considered the search for quality a fad that would soon pass. After all, hadn't we all come through the Management by Objectives era?

Quality is not a fad. It is real and beneficial to any organization practicing good quality management procedures. AISC in the mid-1970s developed a quality check list that if followed would tell our customers that there is a quality procedure in place for fabricators of steel buildings and bridges. We are constantly responding to calls from construction managers, as well as government agencies both federal and state, for lists of AISC certified fabricators. With the increasing demand for quality certification by many of our users, we believe it is important that AISC provides a quality certification program to fabricators of steel buildings and bridges that tells our customers that the fabricators wearing the AISC certification seal have quality programs in place.

Some of our members have not elected to have their policies and procedures certified. This does not mean they cannot do quality work. The certification program makes a positive statement, not a negative one. It does says that those who are certified have elected to have their programs reviewed and inspected by an outside agency to "certify" that their policies and procedures meet certain criteria to produce quality work. The growth in AISC certified plants over the years is testament to the benefits of the program.

The AISC Quality Certification Program fits fabricators of all sizes, yet the perception is that one must have a large facility with a large staff. Of the 282 AISC certified plants, 85% fit the small plant category with 13% classified as medium size and only 2% considered large plants.

Neil W. Zundel

Neil W. Zundel,
President, AISC

STEEL INTERCHANGE

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

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

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

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

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

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

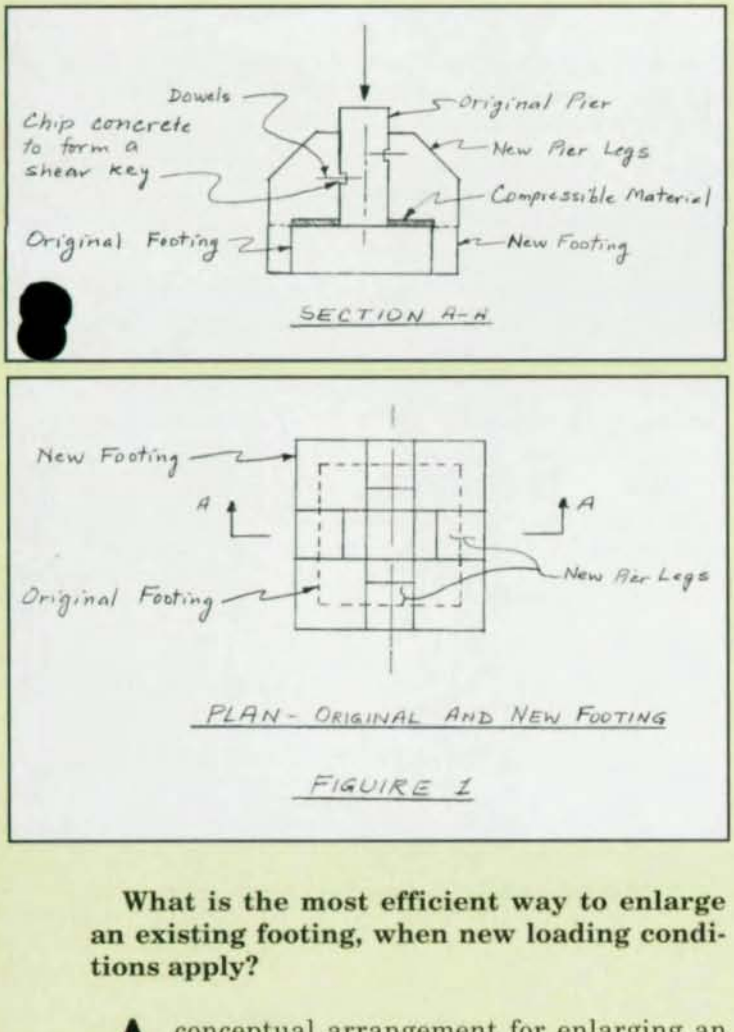


FIGURE 1

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

A conceptual arrangement for enlarging an existing footing for new loading conditions is shown in the accompanying figures (above).

Pier legs carry additional load which is transferred to the new footing by shear keys and dowels as shown. Design bearing pressure underneath both footings should be as uniform as possible.

Needless to say that all loads from the column should be removed before extension of the existing footing takes place.

Vijay P. Khasat, P.E.
Clinton, OH

In what instances, if any, and under what criteria can the attachment of grating with mechanical fasteners be used to provide lateral bracing to the compression flange of the members supporting the grating in applications such as walkways and catwalks?

The American National Standard, ANSI/NAAMM MBG 531-93, *Metal Bar Grating Manual, 5th Edition*, which covers steel, stainless steel and aluminum grating, provides examples of anchorages to use to use in installations where grating is subject to removal. Locations of these clips are suggested also.

This standard also defines the three types of grating as follows:

•**Rivetted**—Grating composed of straight bearing bars and bent connecting bars, which are joined at their contact points, by riveting.

•**Welded**—Grating in which the bearing bars and the cross bars are joined at all of their intersections by either a resistance weld or conventional hand welding.

•**Pressure-locked**—Pressure-locked means bearing bars are locked in position by cross bar deformation instead of riveting or welding. Several proven methods are:

1) Expansion of extruded or drawn tubular cross bar;

2) Extruded cross bar deformed or swaged between bearing bars;

3) Press assembly of rectangular cross bars into slotted bearing bars.

The type of grating as defined above, the relative size of cross bars to bearing bars, the span-depth ratio of the bearing bars, the lack of both theoretical and empirical data, and the possibility of the grating itself being removed, are reasons NAAMM does not encourage the use of metal bar grating to provide lateral torsional support.

Obviously, there is some support provided, but the design provisions have not been established.

The Metal Bar Grating Division of NAAMM is interested in knowing of any documented tests of beams relying on metal bar grating to provide support against lateral torsional instability.

Edward R. Estes, Jr., P.E.
National Association of
Architectural Metal Manufacturers
Chicago, IL

What is the most efficient and cost-effective way to connect a steel wide flange girder to a concrete column?

In the December 1994 *Steel Interchange*, one proposed solution showed a detail with a steel beam extending through the width of the concrete column. Connections similar to this have in fact been used in several composite framed structures in the US and Japan for cases where large bending moments are transferred between the steel beams and reinforced concrete columns. In addition, over the last ten years, there have been numerous tests of such connections conducted in the US and Japan to evaluate the effectiveness of various joint details. For further information and additional references on such connections, readers should consult the following document that was prepared by a task committee of the ASCE Committee on Composite Construction: Guidelines for Design of Joints Between Steel Beams to Reinforced Concrete Columns, ASCE *Journal of Structural Engineering*, August 1994, Vol. 120, No. 8, pp. 2330-2357.

Gregory G. Deierlein
Cornell University
Ithaca, NY

New Questions

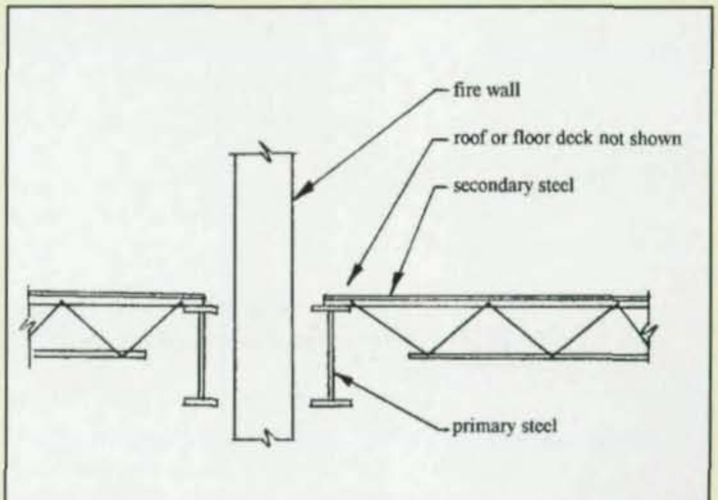
Listed below are questions that we would like the readers to answer or discuss.

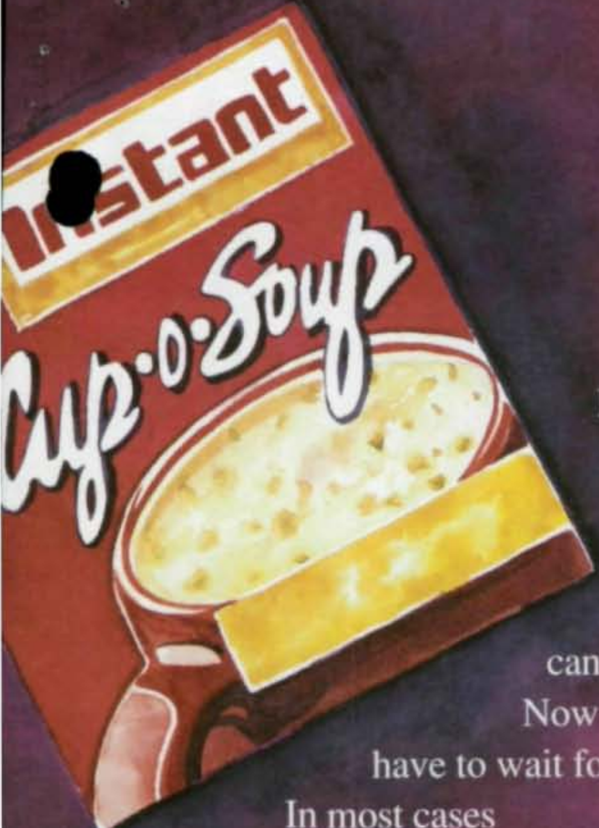
If you have an answer or suggestion please send it to the Steel Interchange Editor, Modern Steel Co nstruction, One East Wacker Dr., Suite 3100, Chic ago, IL 60601-2001.

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

For fire wall construction, building codes say the wall shall have sufficient stability under fire conditions to allow for collapse of construction on either side without collapse of the wall. In a tied fire wall application, a flexible anchor or break-away connection is recommended to laterally stabilize the wall and under fire conditions to let go and not to pull down the wall due to the collapse of the structure on the fire side. What is the optimum detail (effective and economical) for this type of connection? (If a melted anchor is not a preferred option.)

Nai-Jang Wen
Belcan Engineering Group, Inc.
Cincinnati, OH





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CHAPARRAL STEEL.

This bi-monthly column deals with legal matters of interest to designers, fabricators and contractors. We solicit your comments, concerns, suggestions and questions, both as to individual issues and subjects that you would like to see treated in this column. Some of the issues that we plan on covering in the future include: contract provisions; OSHA standards; employment law; alternative dispute resolution; and dealing with the EPA. Comments should be sent to: Modern Steel Construction, One East Wacker Dr., Suite 3100, Chicago, IL 60601-2001.

Construction Arbitration: Avoiding The Pitfalls

FORMAL DISPUTE RESOLUTION PRECESSES, IN LIEU OF TRADITIONAL LAWSUITS, have grown in favor in the United States. Potential litigants perceive the virtues of arbitration to be quicker conflict resolution with lower legal fees. Civil actions in some areas of the country may take more than 10 years before all appellate procedures have been exhausted.

The use of arbitration by architects, engineers, and contractors has another distinct advantage over the typical litigation process. The arbitration panel, which sits as the judge and jury, can be composed of individuals who have expertise in the construction industry. The average judge sitting in circuit court may have absolutely no experience in construction issues, but yet is called upon to settle a complex construction dispute.

The growing popularity of arbitration is demonstrated by the nearly *$10 billion* in claims that have been submitted to arbitration for dispute resolution in the last 10 years. While two-thirds of the claims settled in 1993 were under $50,000, nearly 200 cases were filed in 1993 alone claiming damages in excess of a half a million dollars, according to the American Arbitration Association.

The wide range of the amounts of claims is evidence that arbitration, under the appropriate conditions can be a viable alternative for settling disputes of any size. Although arbitration is a viable alternative to traditional litigation, architects, engineers and contractors should be aware that arbitration can sometimes lead to some unjust and unexpected results. This article will discuss potential problems in the arbitration forum.

Before getting into specific details of arbitration problems, it is worthwhile to discuss the courts attitude in getting involved in arbitration disputes. Most judges will support any method of quick and efficient dispute resolution. Since arbitration accomplishes this goal, judges tend to rule in favor of anything that forces, favors, or upholds arbitration agreements. Keeping in mind that judges will go out of their way to compel arbitration and upholding the arbitrators award, let's look at several issues of concern in arbitration.

Issue 1: Can The Arbitrator Award Punitive Damages?

A prevailing party in a lawsuit can be awarded damages of several different types. One type of award is called compensatory damages. Compensatory damages provide money to the injured party to place them in a position they would have been in if the defendant had not breached a duty. Another type of award is nominal damages in which a small sum of money is given to the prevailing party. Such a small damage award is appropriate in cases where the plaintiff proves injury but cannot prove any monetary damages from this injury.

A type of damage award that is not at all related to the amount of damages suffered by the plaintiff is punitive damages. Punitive damages are awarded by judges who feel that merely paying back the prevailing party is insufficient to deter the defendants egregious conduct in the future. These damages usually entail a significant amount of money awarded to the plaintiff to punish the defendant. The purpose is not only to reprimand the defendant but also to provide a disincentive for others to act in a similar manner.

Since such damages are a punishment, it is questionable if it is appropriate to allow an arbitrator to award punitive damages. It should be remembered that judges are either elected officials or chosen by elected officials and therefore have the public to answer to. This system of checks and balances is not normally present in the arbitration system and it may not be appropriate for arbitrators to be making policy decisions.

The courts have taken three approaches to the allowance of the awarding of punitive damages by arbitrators. These approaches are:

Approach 1-The arbitrator is not allowed to award punitive damages.

Approach 2-The arbitrator can award punitive damages only if the contract specifically allows for it.

Approach 3-The arbitrator can award punitive damages as long as the contact does not forbid it.

With regards to punitive damages, it would be a serious mistake not to specifically exclude them from any arbitration agreement. There is a strong tendency for these damages to be awarded arbitrarily.

Issue 2: Will the Laws of Your State Govern?

Many construction projects draw A/E firms and construction companies from different states. These states will typically have different laws as well as different judicial interpretations of the law. In fact, there is a complex body of law focused solely on the issue of which state's law will be applied when the party's are from different states. These states will typically have different laws as well as different judicial interpretations of the law. In fact, there is a complex body of law focused solely on the issue of which state's law will be applied when the party's are from different states.

The reason for the interest in the governing law issue is that victory in many cases hinges on which state's law will be applied. A clear winner in one state may be a sure loser in another. As an example consider the case of Mastrobuono v. Shearson Lehman Hutton, Inc. 20F2d. 713 (1994), where if Illinois law applied, the

Patrick J. Mazza, Esq., of Patrick J. Mazza and Associates in Chicago, has practiced in the area of construction law for 25 years. He is a graduate of Notre Dame and DePaul Universities.

August Domel, Ph.D., Esq., is senior project manager with Gouwens Engineering Consultants Inc. in Elmhurt, IL, and is an adjunct assistant projessor in the Civil Engineering Department of the Illinois Institute of Technology.

plaintiff would most likely receive a $400,000 award for punitive damages and nothing if New York law applied.

In 1985, Antonio and Diana Mastrobuona opened a stock brokerage account with Shearson Lehman Hutton. Aproximately three years later, the Mastrobuono's filed suit in Illinois alleging that the brokerage firm had mishandled their funds. Shearson Lehman Hutton moved to have the case sent to arbitration, since the brokerage contract contained an arbitration clause. The request was granted and an arbitration panel awarded Mastrobuono $160,000 plus an additional $400,000 in punitive damages.

Shearson Lehman Hutton filed a motion in Federal court to have the punitive damages award nullified. Their position was based on a clause in the contract that stated:

"This agreement. . .shall be governed by the laws of the State of New York."

They argued this position because the State of New York does not allow arbitrators to award punitive damages. The court denied the awarding of punitive damages because it agreed that New York law was controlling.

The court took this position even though the plaintiffs lived in Illinois, the case was filed and heard in Illinois, and the broker was working in Texas. The only connection to New York law was that the contract stated New York law would apply and the home office of the defendant was located there.

In a construction contract, an appropriate clause to be inserted is one that would have the parties agree that the law of the state where the project is located will govern. Regardless of the governing law state, the parties must be aware of the potential problems this situation poses.

ALTHOUGH ARBRITATION IS A VIABLE ALTERNATIVE TO TRADITIONAL LITIGATION, ARCHITECTS, ENGINEERS AND CONTRACTORS SHOULD BE AWARE THAT ARBRITRATION CAN LEAD TO SOME UNJUST AND UNEXPECTED RESULTS

Issue 3: Will Mistakes In The Facts Be Reversed on Appeal?

When a decision made by a judge is incorrect, the losing party almost always can appeal to a higher court. If the higher court is of the opinion that the decision is incorrect, it will either be reversed or vacated and sent back to be retried. This is not the case for incorrect arbitration decisions.

It is well settled law throughout the United States that an arbitration award will not be overturned if the arbitrators made a mistake. The courts have taken this position because they believe that by choosing an arbitrator, the parties have chose the person(s) who will interpret the law and the facts for their case to the exclusion of the court.

Only rarely will the courts attack an arbitration award, and then only when there has been a gross mistake. A mistake may be considered gross when the arbitration award implies bad faith and failure to exercise honest and fair judgement. However, if the mistake does not qualify as a "gross mistake" the losing party must abide by the incorrect decision.

In the case of The Island on Lake Travis LTD (Travis) v. The Hayman Company General Contractors, 834 S.W. 2d 529 (1992), the plaintiff, Travis, made a motion to overturn the arbitrators award for several reasons. One of the reasons for the appeal was that Travis claimed that the method for calculating damages was incorrect. The Appellate Court of Texas concluded that the arbitrators decision was "not so erroneous" as to be considered a gross mistake. The court proceeded to confirm the arbitrator's award even though the court admitted the decision was erroneous.

The Uniform Arbitration Act does not expressly recognize any remedy for an arbitration mistake, such as an improper interpretation, a misapplication of the law, or an error in logic. If a major issue may be involved, an insertion of a clause in the contract stating that the rules of evidence of a particular state will govern and the parties agree that either party may appeal the findings to the circuit court may be prudent.

Issue 4: Will Mistakes in the Application of the Law be Reversed on Appeal?

In Issue 3, it was discussed whether a mistake of fact by the arbitrator could be overturned on appeal. A similar problem arises when the arbitrator has made a mistake in the application of the law. It also leads to a similar result. Courts will only overturn an arbitration award if there is a gross misapplication of the law.

Consider the case of Perini Corporation v. Greater Bay Hotel & Casino, Inc. 610 A.2d 364 (1992). Perini agreed to be construction manager for the renovation of the Greater Bay Hotel in New Jersey for a fee of $600,000. The parties went to arbitration to settle disputes that arose from the project. The arbitrator awarded $14,500,000 in damages to Greater Bay Hotel and Casino.

Perini argued on appeal to the Supreme Court of New Jersey that it was a mistake to award damages for lost profits after the date the project was substantially completed. The Court stated that Perini's position was amply supported by previous case law as well as by the practice in the construction industry. Furthermore, the Court also said that it appeared that even the plaintiff agreed that the project was substantially completed on the date in question. Nevertheless, the Court would not overturn the arbitrators decision.

The discussion in this article listed four potential problems that can occur when arbitration is utilized. These problems are but a few of the wide range of possibilities. It should be noted that the authors of this article by no means desire to disparage arbitration which has proven to be successful forum in which parties can settle their problems in a less adversarial atmosphere with those whom they may desire to continue a business relationship with in the future. However, careful drafting of arbitration agreement is essential to provide an arbitration hearing that will produce fair results.

01-034

BOOK REVIEW: *STEEL STRUCTURES—CONTROLLING BEHAVIOR THROUGH DESIGN*

NEW TEXT ON STEEL PROMOTES "DUCTILE" THINKING

By Robert F. Lorenz

ROBERT ENGLEKIRK, PH.D., S.E., A NOTED WEST COAST STRUCTURAL CONSULTANT AND EDUCATOR, has authored a new steel textbook, Steel Structures—Controlling Behavior Through Design. Although written just prior to the Northridge earthquake, it has become an unusually timely document since it includes a wealth of information that helps explain the ductile response of steel framing to earthquakes. The book should literally add new meaning to our understanding of ductility in general and should provide some new tools to evaluate the inelastic performance of a given steel frame.

The first thing you will notice inside this book is that it only has six chapters. The first three chapters cover the familiar ground of materials, flexure and stability in a rather non-traditional way. The last three chapters are unique, and although sometimes burdened with some repetitious analytical technique, they do present practical procedures of breaking down the design process into a series of rational decision making steps. I found the chapter entitled "Behavior of Bracing Systems," a real breakthrough in explaining the dynamic behavior of buildings in terms that non-seismic designers can understand.

Englekirk is quite critical of the current effect of "prescriptive" codes on seismic designers as it inhibits creativity and true understanding of the design process. Certainly, some of his ideas, such as "plastic deformation" criteria for establishing critical design limits, will be controversial, but it should be welcomed by those who realize much code-language has become too strength-oriented and loaded with time consuming paraphernalia and detail.

Educators should carefully review this text since it may be a forerunner in tone of future textbooks that would place deformation right up there with strength in explaining the latest way steel frames perform.

This book is unique, as it depends on the reader accepting a new, conceptual approach to steel design. In other words, it is not a quick read. On almost every page, you are obliged to stop and think. And because of that feature alone, there is much to recommend about this book.

More information on Steel Structures—Controlling Behavior Through Design can be obtained from John Wiley and Sons, Inc., 605 Third Ave., New York, NY 10158-0012; fax (212) 850-6088.

Robert F. Lorenz is director of education and training at AISC.

STEEL BRIDGE INDUSTRY GAINS UNIFIED VOICE

FOLLOWING A YEAR OF INTENSIVE PLANNING, THE NATIONAL STEEL BRIDGE ALLIANCE (NSBA) has been formally organized. The NSBA joins together the nation's steel bridge fabricators, the steel producers and other groups serving the bridge industry—its owners, designers and contractors.

The NSBA is dedicated to aggressively serving the needs of the bridge industry by providing state-of-the-art design technology, developing and encouraging the use of new construction technologies and the best use of high performance steel products. The NSBA is sensitive to the resources available to the State Departments of Transportation to repair, replace and construct new bridge structures in order to maintain this country's excellent highway system. To that end, the NSBA will provide the best of current design and construction practices to the states' engineering departments, construction departments, their consultants and to industry.

The National Steel Bridge Alliance membership is open to all organizations interested in the development, promotion and construction of cost-effective steel bridges. To accomplish its objectives, the Alliance will include both a national organization and a network of regional steel bridge fabricator groups.

The NSBA will function as a division of the American Institute of Steel Construction, Inc. The governing body of NSBA will be a Steering Committee consisting of steel producers and steel bridge fabricators. Robert P. Stupp, Stupp Bros., Inc., was elected Chairman with Mike Martin, Bethlehem Steel Corporation, to serve as Vice Chairman. Neil W. Zundel, President, AISC, was appointed NSBA Director.

For more information on The National Steel Bridge Alliance, please contact Neil W. Zundel at 312/670-5401.

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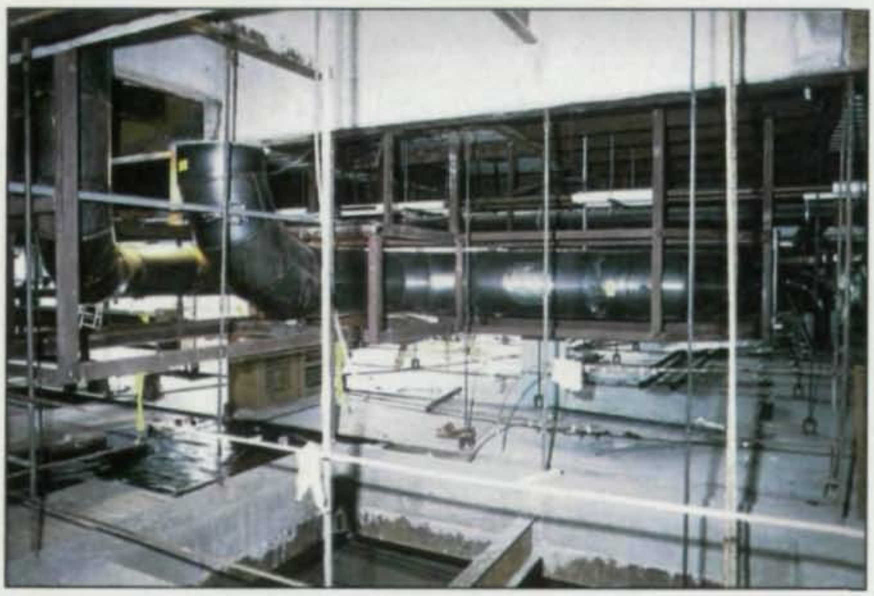
Interstitial Floor Aids Hospital Design

A modular mechanical and electrical system coupled with an insterstitial floor design provided maximum flexibility

Photo by Balthazar Korab (Korab/Hedrich-Blessing)



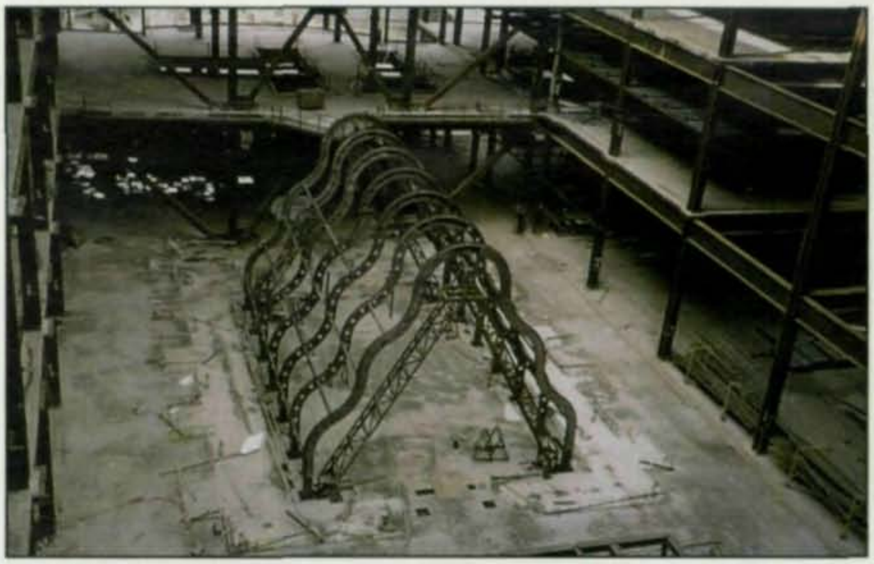
By John McCarthy, P.E.

A HOSPITAL'S COMPLEXITY MAY BE SECOND ONLY TO A MODERN SPACE CRAFT. With 18,000 tons of steel spread over 1.2 million sq. ft., the new Veterans Administration Medical Center in downtown Detroit is truly a flagship commanding respect. The $229 million hospital is the largest single construction contract the VA has ever awarded and, in today's climate of shrinking dollars, may be one of the last big hospitals.

The 503-bed hospital complex is divided into three major sections: a seven-story medical/surgical nursing tower, a five-story psychiatric nursing tower, and a four-story diagnostic/treatment block including a radiation oncology center, an emergency center, and an ambulatory care center. In addition to the medical buildings, the development includes an energy center and two parking decks. The 19-acre site spans three city blocks in the Detroit Medical Center.

The design and construction complexities inherent in such a large project demanded the skills of a top-flight team. Smith, Hinchman & Grylls Associates, Inc. (SH&G), a national, full-service architectural and engineering firm won the assignment to design and engineer a facility to replace the Allen Park VA Medical Center (circa 1939) in the mid 1980s. J. W. Bateson, of Dallas, Texas, and R. E. Dailey & Co. (now the Perini Co.), of

Southfield, Michigan joined forces to construct the hospital. Bateson brought its considerable history of VA type construction and Dailey added its own large project experience and local knowledge to the team. Bateson/Dailey did an enormous amount of pre-investigation before pouring concrete or setting steel. This resulted in a smoother than usual construction period. Most of the problems were identified on paper and evaluated prior to the actual construction. The partnering agreement, formed at the beginning of the project, facilitated open communication and effective problem-solving among all the parties during construction.



Key to the success of the new Veterans Administration Hospital in Detroit is the use of an interstitial floor (opposite page). Another interesting feature is a large chapel in the center atrium (above and left).

Interstitial Floor

The entire facility is based on a modular mechanical and electrical services system coupled with an interstitial floor design to provide a maximum degree of flexibility for future changes in healthcare delivery. The VA introduced the interstitial floor in 1969. Interestingly, Bateson was the contractor on the first hospital interstitial job and has probably built more interstitial space than any other contractor. The interstitial floor is intended to speed construction since the mechanical and electrical trades can begin to route the major distribution systems throughout the hospital without having to closely coordinate the finishing contractors on the functional floors below. Future flexibility is increased because changes can be made to the distribution systems while the rooms below are in use, with only the final hookups requiring a room to be shut down.

The interstitial floor consists of $1^1/_2$-in.-deep metal roof deck which is fitted in between 6-in. purlins hung from the floor beams above with $^3/_4$ or 1-in.-diameter rods. The hospital has about 16,500 hanger rods. A walking surface of five inches of very low density concrete (200 psi) is placed over the metal roof deck. The details at the expansion joints and at the interface of the mechanical rooms caused the most difficulty since there is hardly any stiffness in the system.

To increase the flexibility of the facility even more, almost the entire floor area is covered with a second concrete pour of light weight concrete. This provided two benefits: future flexibility for modifications in healthcare delivery and a very flat concrete floor finish. The flatness of the floors was increased

because the initial deflection of the composite system took place with the structural slab and the additional deflection of the topping pour was minimal. As a result, the case work required less shimming and door heads line up. This had been a problem for the VA on previous projects.

GLAZED BRICK AND LINTELS

The most striking feature of the hospital is its vivid exterior color. Nearly 3.5 million glazed bricks—teal for the nursing unit, burgundy for the psychiatric unit, cream for the diagnostic unit—will glisten under the sun. Though glazed brick is not common in Michigan's climate, SH&G's in-house masonry expert worked with the contractor to produce a product that should pass the test of time. Indeed, the glazed brick-clad exterior of the landmark General Motors Technical Center in Warren, Michigan, designed by Aero Saarinen and co-architects SH&G, is as bright and durable today as it was 40 years ago.

The VA hospital's masonry cladding dictated a stiff and relatively deflection free spandrel system and a construction sequence that required the concrete floor and topping slabs be poured prior to placing the masonry on the lintel system. The exterior wall of brick and block was supported on a lintel system that wrapped the buildings at each floor. The typical lintel system consisted of an 8-

in.-by-8-in. tube to support the block and a protruding $^{3}/_{8}$-in. plate to pick up the brick. This system was hung from the floor above, braced laterally at the interstitial deck and fastened to the columns with adjustable connections to allow very fine adjustments in locating the lintel for the masonry.

Water Tunnel

About a third of the way through the design of the hospital, engineers discovered a subsurface surprise: a 12-ft.-diameter water tunnel directly under the structure. The tunnel, which supplied water for over a million people, demanded imaginative problem-solving. At issue was the impact of the building excavation and foundation loads on the water tunnel and the possibility of a tunnel collapse and its impact on the structure. Our team's solution? To design a grid-like bridge system of deep concrete beams that were supported on deep drilled piers reaching the hardpan 100 feet below. The drilled piers were designed to withstand the highly unlikely event of the water tunnel collapsing and washing away the soil below the hospital. The grid was composed of 12-ft.-deep grade beams that ranged in width from 10 ft. to 28 ft. The heavily reinforced concrete beams required massive concrete pours that began in the evening and would last throughout the night. The large pours required between 200 and 250 concrete truck loads per segment and more than 1,900 tons of epoxy coated reinforcing.

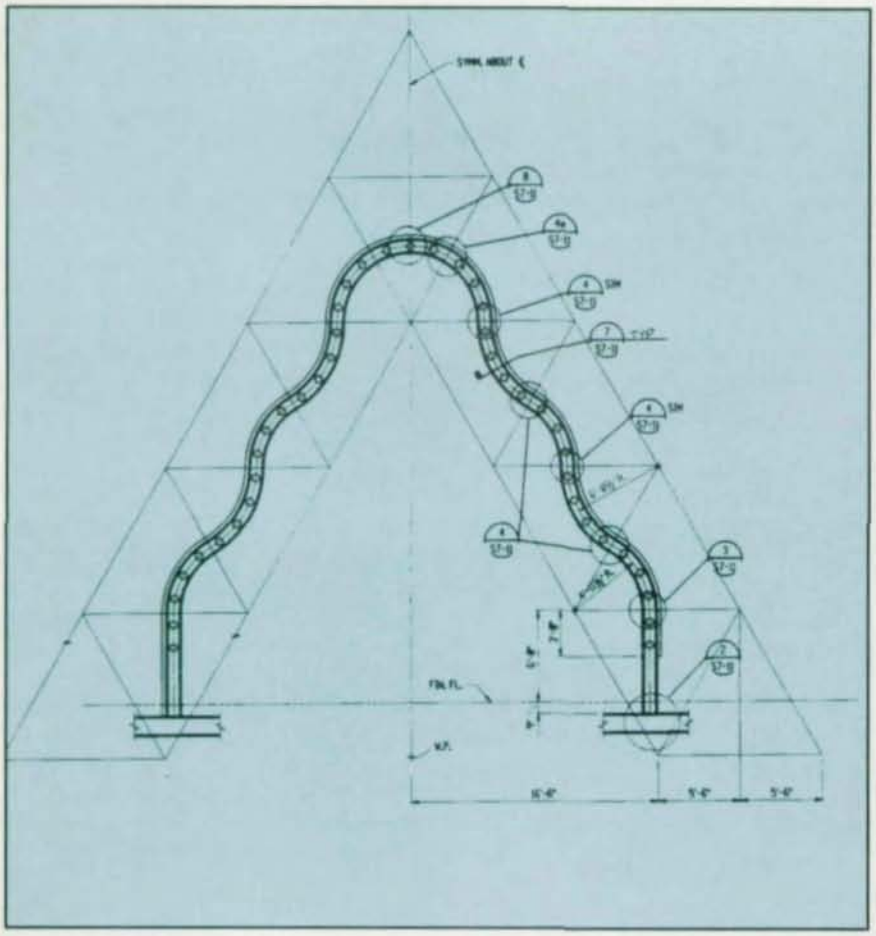
Atrium

The three medical buildings are connected by a four-story glass-topped atrium that runs nearly 500 ft. in the North South direction. The central atrium creates a visually exciting space for patients, visitors and staff. Covering the atrium is a curved skylight over 80-ft. above the floor. Of special concern to the structural engineer was the fact that the skylight is connected to two separate buildings with differing seismic displacements. The solution was to design a three-hinged arch, which allows the buildings to move in three directions without distress. The top connections use three inch diameter pins and the connection at the base uses 4-in.-diameter pins. The arches are designed to accommodate up to seven inches of differential shearing movement between the buildings.

Adding to the visual excitement is a three-legged fountain and two buildings tucked into a cut out in the atrium where the Diagnostic and Treatment Block floor was removed. The travel agency and social services buildings are semi-circular in shape and the multi-leveled structures support two levels of glass block at the interface of the roofs and the upper walls.

The atrium also houses an undulating glass block chapel, the design of which is intended to elicit the form of a water fall. The glass block forms a concave and convex wave as it curls down over twenty feet from peak to floor. Spanning between steel frames that follow the outline of the glass block, the block was reinforced with tees at the points of inflection. The frames were fabricated of plates cut or bent to a shape that resembles a W12, except for the 6-in.-diameter holes that were drilled at regular intervals along the length. The frames were braced at the ends by triangular space frames constructed of steel tubes to increase lateral stability and resist seismic forces. These space frames were hidden in the triangular architectural features.

The structural fabricator, AISC-member SMI-Owen Steel Co. Inc., faced many difficult challenges during the fabrication of this job. The largest challenge was the small area that the gen-



eral contractor allowed to stage the structural steel on the site. According to Tony Hazel, chief engineer of SMI-Owen Steel, the on site storage was limited to 30 trailers. To facilitate coordinated delivery of more than 1,000 trailer loads of material, SMI-Owen Steel set up a separate marshaling yard at their Columbia, South Carolina facilities, which allowed the company to stage sequenced loads of structural steel well in advance of the site erection operation for quick loading as deliveries were required. Close coordination with the structural steel erector Dumas-McGuire Group Services, Inc. of Detroit resulted in a smooth flow of material to the site. David Ryepka of Bateson/Dailey notes that despite the record setting rainfall during the erection period and the tight site, the steel erection was completed within weeks of the original nine month schedule.

Framing Solution

Since the floors included a 3-in. light weight concrete topping, an interstitial floor and a brick and block exterior wall, the mass of the building was higher than normal. The interstitial floor resulted in floor to floor heights exceeding 21 ft. For these reasons—and the fact that the seismic code requires an importance factor of 1.5 for hospitals—the seismic forces governed the lateral design. Due to the high floor to floor distance, drift was the deciding factor in the selection of member sizes.

Due to programming constraints and the requirement to keep the facility as flexible as possible for future medical advances, workable solutions to lateral load systems were limited. The three pieces of the medical complex were divided and treated separately. The Nursing Tower and the Psychiatric Tower allowed bracing to be located in the elevator core areas and at

the ends of the buildings. Even though the major mechanical systems were separated between the buildings, there still were some major ducts running through the bracing between the buildings. In the long direction of these buildings, two lines of moment frames were utilized. Because some of the columns were detailed as part of the moment frames and the braces, the column and splice details were complicated.

The diagnostic and surgery block had its own set of problems. Here the flexibility and program requirements dictated that only a moment frame could be used. The frames were distributed throughout the structure to minimize the required sizes. The possibility of two additional future floors required rather large members for the columns and girders.

Types of Connections

Single angle connections were suggested by the fabricator during the detailing stage of the project. The beam to column moment connections were the typical flange welded and column web stiffeners with bolted webs. The column splices were originally detailed on the contract documents as full penetration welded connections. The fabricator proposed to use fully bolted connections, which proved to be cost-effective, even though they added about 100 tons of fabricated material to the job. Tony Hazel of SMI-Owen Steel believes that reducing the amount of field welding was critical since there already was an abundant quantity of field welding required for the project. Bolted connections also shortened the schedule, since the enormous amount of preheat required for the large column sections would have slowed down construction. Also, bolting the column splices reduced the concerns of welding jumbo sections, since some of the bracing columns were in tension.

The hospital is fully sprinklered and the two-hour fire rating is provided by the interstitial deck system. Therefore, the functional floors did not need to be one of the standard $1\frac{1}{2}$- or two-hour rated systems. This opened up the possibilities for floor systems tremendously. About seven different structural floors systems were investigated for economy and future flexibility. The systems ranged from concrete frame with wide pan joist systems to precast concrete planks on a steel frame to posttensioned structural systems. Two and three inch composite floor decks and grade 36 and grade 50 steels were also investigated. The system that proved to be most cost effective was the 3-in. metal deck on high strength steel utilizing partial composite action.

A project the size of the VA hospital includes just about every type of structural material. Joists were used to support the roof deck over the electrical substation. The joists were supplied by Owen Joist Corp. The deck supplier was Bowman Metal Deck. The deck varied in depth from $1\frac{1}{2}$ to 2 to 3 inches. The concrete used to fill decks varied in thickness depending on the location and load requirements. This resulted in nine types of slab systems of differing deck depths, concrete thickness or reinforcing.

During design, SH&G performed a value engineering analysis. The painting of the structural steel was deleted at an estimated savings of more than $100,000. Only exposed structural steel was painted or galvanized.

Computer Programs

The structure was designed using several computer programs including GTStrudl for major items such as the lateral loads and the grade beams over the water tunnel. For some of the smaller, but no less complex, framing systems including the chapel, sacristy and curved beams, STAAD III was used. The composite beams were designed using a program developed by SH&G. Spread sheets were created to analyze and design the column splices, beam to column moment connections and beam framing connections.

John McCarthy, P.E., is a project structural engineer with Smith, Hinchman & Grylls Associates, Inc., in Detroit.

Innovative Design Emphasizes Easy Access

Lancaster General's new Health Care campus is organized around a central atrium, creating an easily understandable traffic flow for patients, visitors and staff

By John A. Westenberger, PE

When Lancaster General Hospital in Lancaster, PA, opened its brand-new Health Campus in September, it exemplified the latest trends in health care design, most notably the move towards a better organizational structure that helps minimize patient and visitor disorientation. The design of the 350,000-sq.-ft., three-story ambulatory care center, known as The Health Pavilion, emphasizes patient comfort and convenience, while also providing physicians and patients with convenient access to outpatient medical care. Unlike many other facilities that contain a maze of confusing corridors, all department waiting and reception areas open to a single interior concourse.

The ambulatory care facility was built on a 121-acre site three miles west of Lancaster General Hospital's main hospital. The site was selected based on the hospital's desire to build a free-standing, off-site outpatient facility in close proximity to the main campus. The site is large enough to accommodate growth over time as identified in the 101-year-old institution's long-term master plan.

The exterior of The Health Pavilion takes cues from the traditional architecture of Lancaster, while also promoting an identity of its own. Facades incorporate Flemish bond brick accented with limestone trim to provide a sense of tradition and permanence.

Inside, a three-story, skylit interior "concourse" is the building's primary organizing feature. This concourse, with its subtle colors and unique architecture, orients visitors and provides easy access to all clinical departments and physician practice suites. Wood wainscoting, ornamental arches and balustrades and granite flooring all contribute to the function and ambience of this public space. Fountains provide interesting focal points and soothing background sounds.

All department waiting and reception areas open to this interior concourse so visitors never have to wander through a maze of corridors. Changes in wall finishes and familiar furnishings such as paneled reception desks and library lamps signify the waiting areas for easy orientation of patients and visitors. These areas are treated with a warm, residential palette of colors and materials. Where appropriate, waiting areas are screened for privacy and comfort.

An express testing area is located adjacent to the lobby area. This first floor also features a Diagnostic Imaging Center; Women's Imaging Center; Physical Rehabilitation Center; and an Eye Center. A one-story Radiation Oncology Center has its own entrance on the south side of the building. Throughout, a separate system of corridors gives staff and supplies access to the "back door" of each department, completely segregating them from patient and visitor traffic.

The second floor houses a Surgical Center. Patient comfort and privacy are emphasized on this floor, which features private spaces for changing and recovery.

A cafeteria-style dining room for visitors and staff ("Today's Taste Cafe"), Renal Dialysis Center, Endoscopy Center, Cardiac Rehabilitation Center and The Wellspring occupy the third floor.

The basement houses general support functions including housekeeping, supplies, storage, staff lockers, offices for plant engineering and building management, mail, waste management, recycling and a loading dock.

Two medical office buildings —housing 19 physician practices —project perpendicularly from each leg of the "L-shaped" Health Pavilion. Outside, large pitchedroofs, brick with limestone trim, glass and metal windows create a traditional feeling. Each medical office building has a separate driveway and covered entrance from adjacent parking areas as well as access to The

MAKING THIS BUILDING S
WOULD GIVE MOST CO

But not Vulcraft. We saw it as one of our greatest challenges ever. Because we not only supplied steel joists and joist girders for the project, we also helped design the framing system so that only limited structural damage could be expected from an earthquake measuring up to 7.5 on the Richter scale.

The Evans & Sutherland building, which contains millions of dollars worth of sensitive computer equipment, is located within a mile of the Wasatch Fault in Salt Lake City.

AFE FROM EARTHQUAKES MPANIES THE SHAKES.

That was essential because the building, which was constructed for Evans & Sutherland Computer Corporation, is located within a mile of the Wasatch Fault in Salt Lake City. What's more, Evans & Sutherland is a leading designer of special-purpose digital computers, software systems and display devices — products extremely vulnerable to damage from seismic tremors.

To plan for maximum protection, Vulcraft was asked to join with the architects and engineers at the design stage of the project. Already, they'd decided to use a "base isolation" system, the most advanced buffering method available. But using our steel joists and joist girders was also an important decision. The joists and joist girders are much lighter in weight than wide flange beams, so the entire building required less steel, lighter columns and less foundation. And this not only lightened the load for the base isolators, it saved appreciably on building costs.

Throughout construction, Vulcraft remained constantly involved, tailoring our delivery of materials to the exact erection schedule and meeting deadlines without fail. What's more, our joists and joist girders helped the steel erectors meet their deadlines. That's because our products are fast and easy to erect — a fact that saves time and money on virtually any job where they're used.

The design team chose a "base isolation" system to accommodate seismic loads. They also specified Vulcraft steel joists and joist girders for the building.

So whether you need Vulcraft's help to protect your building from earthquakes or you want to stay out of the hole when it comes to construction costs, contact any of the plants listed below. Or see Sweet's 05100/VUL.

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P.O. Box 637, Brigham City, UT 84302 801/734-9433; P.O. Box 100520, Florence, SC 29501 803/662-0381; P.O. Box 169, Fort Payne, AL 35967 205/845-2460; P.O. Box 186, Grapeland, TX 75844 409/687-4665; P.O. Box 59, Norfolk, NE 68702 402/644-8500; P.O. Box 1000, St. Joe, IN 46785 219/337-1800. *Architect: Erlich-Rominger Architects AIA; Structural Engineer: Reaveley Engineers & Associates, Inc., General Contractor: The Bettilyon Corporation; Steel Fabricator: D&H Steel Supply, Inc.; Steel Erector: Steel Deck Erectors, Inc.*

Please circle # 37

Health Pavilion's interior concourse for patient and staff convenience. The complex is designed to accommodate two additional medical office buildings; one has just begun construction.

Lancaster General Hospital emphasizes the important link between private physicians and the institution by offering doctors close proximity to diagnostic and treatment space.

These medical office buildings strategically house doctors near their adjacent area of expertise in the ambulatory care center. By placing doctors closer to the process, patients are provided with the finest, most accessible outpatient health care services possible.

EARLY PLANNING

Planning of the Health Campus began in 1989 during the hospital's negotiations to acquire the site. Initial geotechnical investigation of the site revealed sinkhole activity. As a result, a system of strip footings three feet thick and up to eight-ft.-wide was designed to span a nine-ft.-diameter sinkhole at any location.

While the building foundations were excavated, decisions about the design of the facility's structural system were made. Steel was selected for its (1) light weight, (2) speed of erection in the winter months and (3) ease of modification during the fast-track design and in the future.

The floor system consists of A36 rolled shapes, primarily W18s and W27s, supporting $3^1/_4$ in. of lightweight concrete slab on two-inch galvanized composite floor deck. The roof structure also is fabricated from rolled shapes — since the sizes and locations of the 16 air handling units were not firmly established until the subsequent bid package was issued.

The atrium space utilizes a clerestory roof to bring natural light into the space. At the midpoint of the atrium, the floors open for three stories. These focal points are faced with three-story glass walls. Semicircular interior pools featuring waterfalls provide a pleasant setting and soothing background noise.

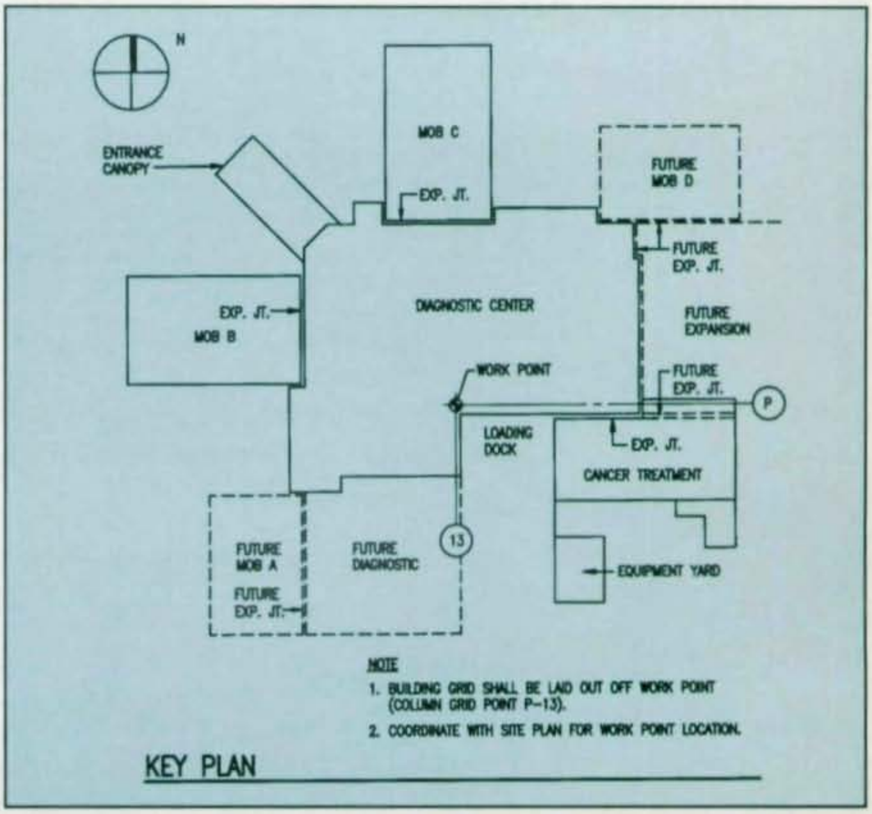
Since these open areas were to remain free of any cross beams, the opening of these interior spaces resulted in 60-foot interior spans. The incorporation of the glass wall required the addition of a horizontal truss into the roof structure. The truss, which is made up of L3x3 and L4x4 shapes, was placed between the glass facade and the clerestory roof to minimize deflections of the skylight. STAAD3, from Research Engineers, was used for lateral load analysis of the entire structure and the design of these horizontal trusses.

Behind the building's main entrance, the floors are also open for three stories. The resulting space is faced by a three-story glass wall and contains two glass elevators. These elevators descend behind the main reception desk. Fully welded moment connections between selected beams and their supporting W14x211 columns provide the necessary rigidity in the glass wall.

The southeast corner of the first floor contains magnetic resonance imaging equipment. To avoid the problems this equipment can experience with steel structures, this area is a concrete flat slab supported on a perimeter steel frame. This section — and the cryogen storage room located adjacent to it — are designed for the actual weights of the purchased equipment. Two fabricators were involved with the project, including AISC-member Stewart-Amos Steel Inc.

Construction Sequence

While the main building was under construction, the design of the entrance canopy, Radiation Oncology Center and medical office buildings was underway. The entrance canopy, which measures 39 ft. by 58 ft., is topped by skylights that extend the entire length of the building.



The clear span of 46 ft. over the driveway allows several vehicles to be parked simultaneously with their doors open. This long span, when combined with the thrust of the skylights, required the use of two more horizontal trusses, made up of L3x3 members, to reduce the Y axis bending of the main structural members. The analysis of this roof was also conducted using STAAD3.

The one-story Radiation Oncology Center houses two linear accelerators and support spaces. The slab-on-grade of this facility is located at the first floor elevation of the main building. Therefore, the linear accelerator concrete vaults are supported on one-story-tall piers, and a retaining wall was constructed adjacent to the loading dock of the main building.

The roof framing here ties into these vaults and frames out additional skylights over the patient waiting area below. Since these skylights are inboard of the roof edges, no special framing was required to resist the thrust forces they impose. Foundations for a future third linear accelerator vault were constructed to facilitate expansion.

The design of the medical office buildings was also an involved process. The initial plan placed two steel-framed office buildings side-by-side over a connecting parking garage constructed of reinforced concrete. A grass courtyard was to be constructed on the roof of the garage. A pair of four-story medical office buildings would be built on the north facade, and another pair of three-story buildings would be erected on the west facade. The cost of this initial design, however, was prohibitive.

The next iteration separated the medical office buildings into four separate structures labeled MOB A, B, C and D. This permitted the two medical office buildings located closest to the main entrance (B and C) to be constructed first. The floor structure consists of ASTM 572 Gr. 50 W18x35 beams and W24x68 girders, which support three and one-quarter inches of a lightweight concrete on three-inch galvanized composite deck. Composite slabs with shear studs were not used so maximum flexibility of the completed structures could be maintained. A fan room located at the roof level is topped with a 12:12 pitch sloped roof surface. To accommodate the snow load build up this sloped roof produces at the main roof level, all roof surfaces are framed in rolled shapes instead of joists. The roof framing consists of W16x26 and W18x35 members.

As the design of the medical office buildings progressed, new design features were incorporated. A palladium window was added to one facade at the elevator lobby location in each building. A radiused C12x20.7 channel was used as a header for this window and was suspended from the roof above.

A more sweeping change involved a drive-through entrance for each medical office building. Due to the roadway radii required, a building column was offset at the lowest story of each building. This resulted in a transfer girder in each facility, one supporting two stories in MOB B and the other supporting three stories in MOB C. Fortunately, W36 rolled shapes—a W36x230 at MOB B and a W36x280 at MOB C, were sufficient to meet the requirements for these members. This avoided the fabrication of plate girders which would have been a problem because the member height was also restricted by the clearance required at the drive-through area.

Following five years in planning, design and construction, The Health Pavilion opened to universal acclaim this past September. Lancaster General Hospital has successfully moved outpatient facilities to the facility. The two medical office buildings originally constructed are fully leased, and Ewing Cole Cherry Brott just completed the design of MOB D. Construction of this building, which will be located at the northeast corner of The Health Pavilion, recently began and is expected to be completed by the end of the year.

John A. Westenberger, PE, is a principal and structural engineer at Ewing Cole Cherry Brott, the Philadelphia-based architectural, engineering, interior design and planning firm that designed the Lancaster General Health Campus. Ewing Cole Cherry Brott is nationally recognized for its award-winning designs of health care, research and development, academic, spectator sports/recreation and senior living facilities.

Solving A Connection Dilemma

The use of a straddle-beam tree column provides for beam continuity around the column rather than through it

By Robert L. Boehmig, P.E.

Interest in beam-column moment connections has been revitalized as a result of last year's Northridge Earthquake, which demonstrated the vulnerability of welds in conventional connections during seismic events.

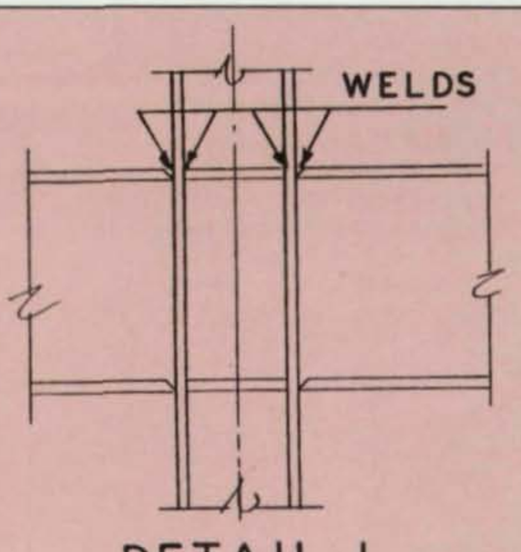
In a conventional connection (Detail 1), beam flange forces flow across the column through as many as four welds. These forces pass through the column flanges in the "Z" direction of rolling—which has tended to cause problems.

In braced frames, most of the moment travels from beam-to-beam, with a smaller component into the column. In moment resisting frames, however, a significant part of the flange force is transferred to the column.

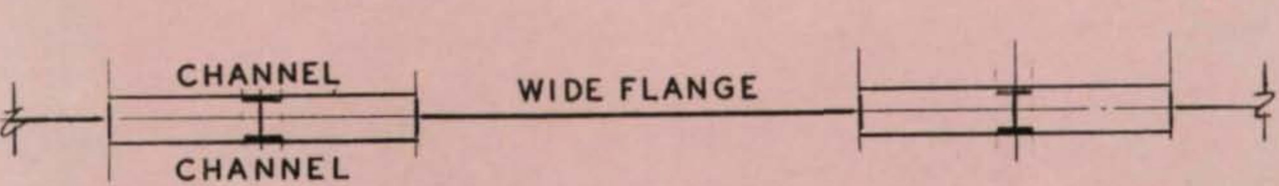
The following proposal (Plan 2) suggests a beam composed of a pair of channels straddling the column and shop welded to it. The channels extend to a point of minimum moment where they are joined in the field to a wide flange section.

The channels are selected for the usually greater negative moment, while the WF beam is sized for the usually smaller positive moment, and can be a slab-beam composite.

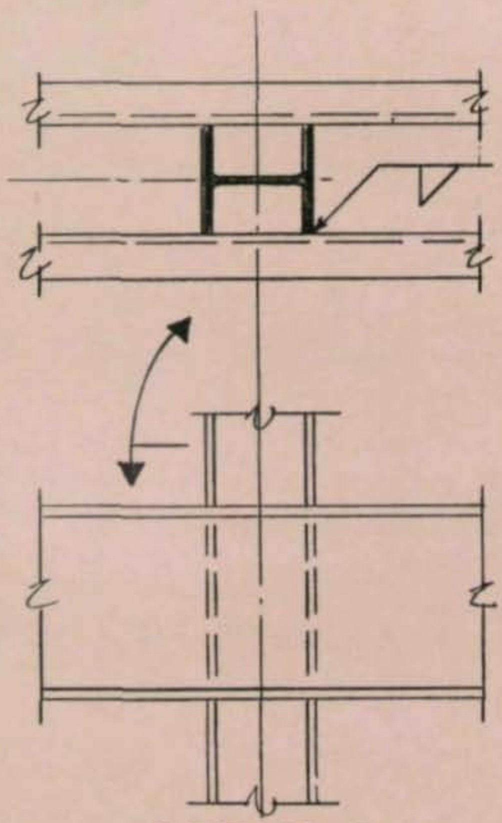
Fillet welds can be used throughout and the field assembly is simple: the WF beam is slid horizontally into place, web bolts are installed, and down hand fillet welds complete the connection.



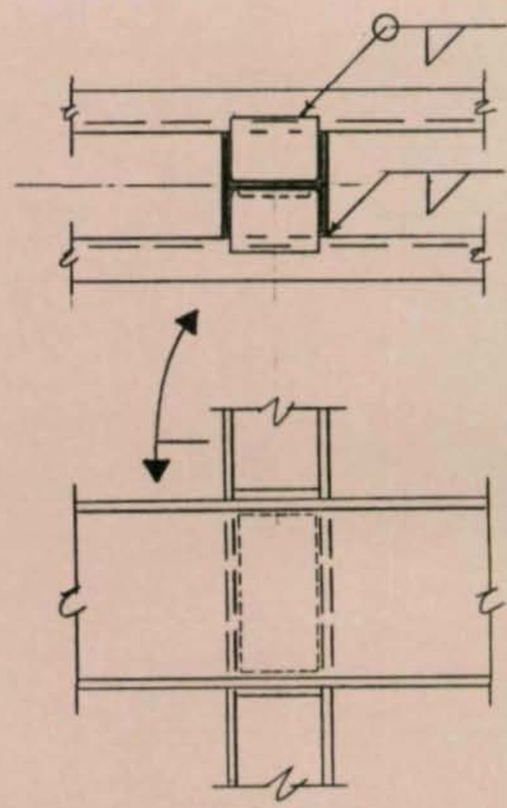
DETAIL 1



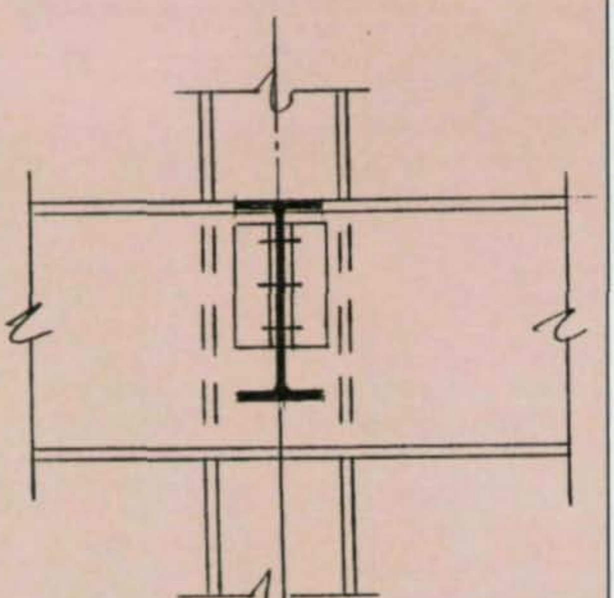
PLAN 2



DETAIL 3



DETAIL 4



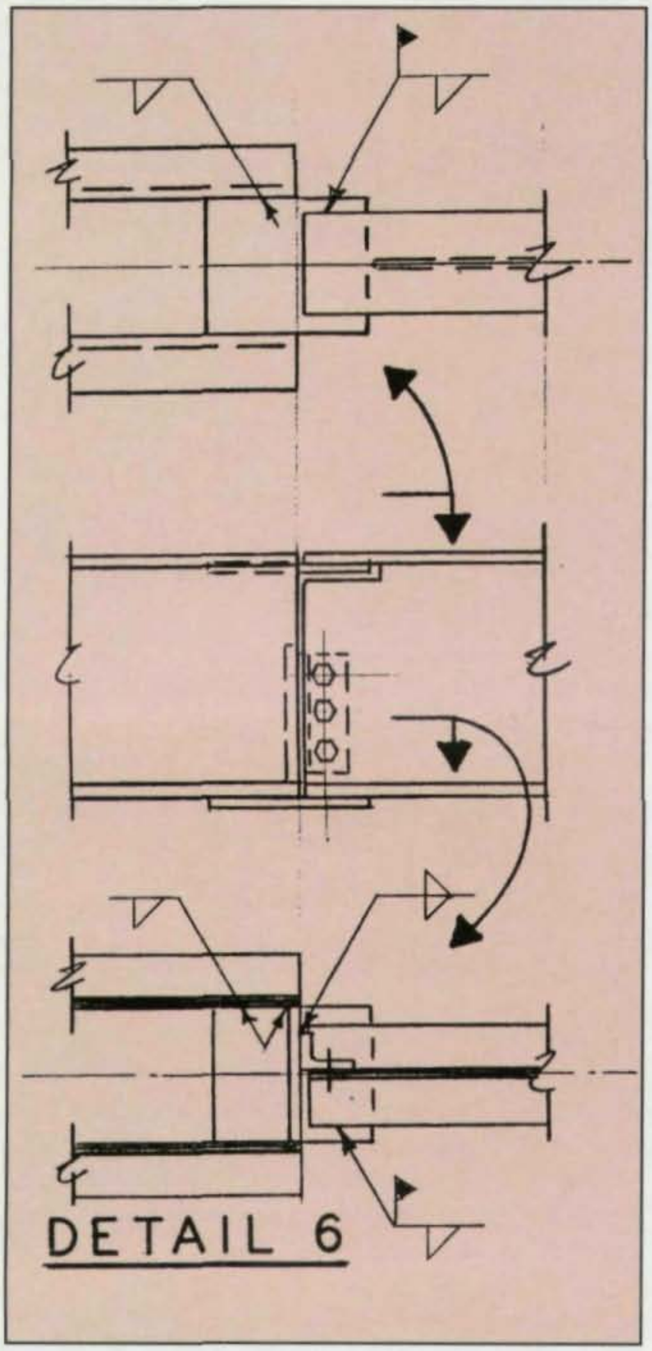
DETAIL 5

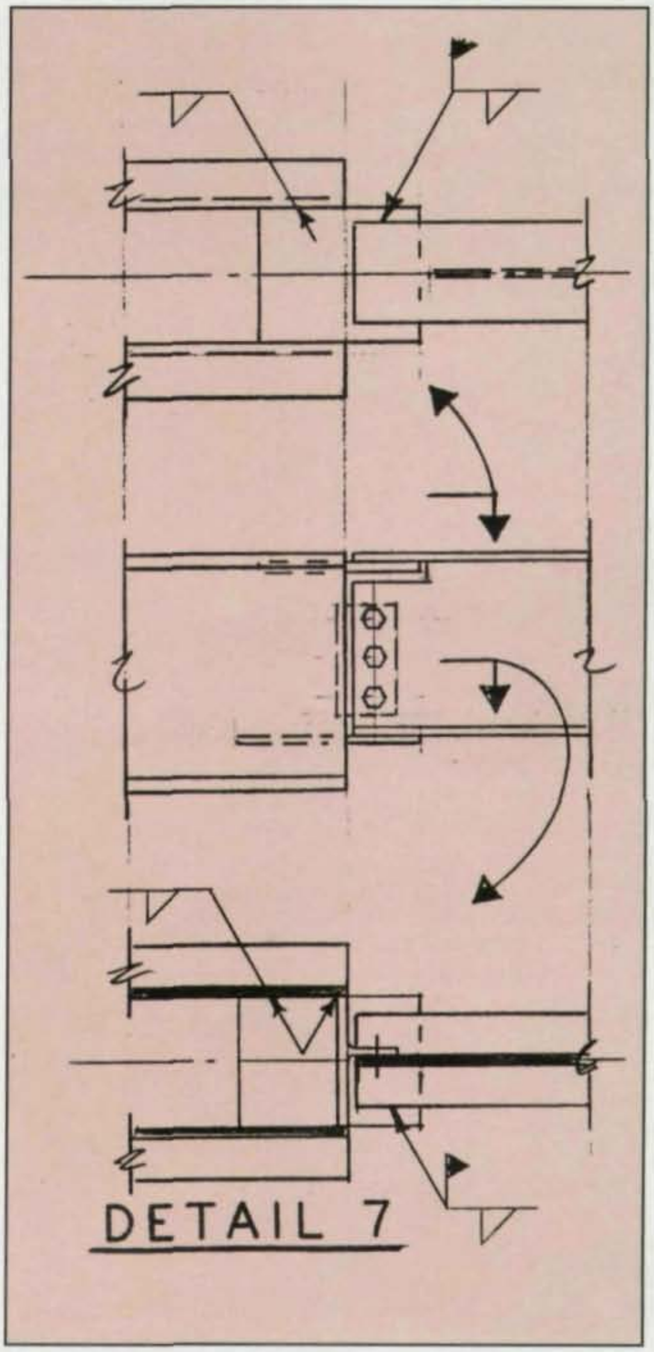
CHANNELS-TO-COLUMN CONNECTIONS

Where the beam to column moment is small, the shop work consists of welding the channel webs to the column flanges (Detail 3).

Where the column moment is significant, stiffener plates and column web reinforcement can be provided (Detail 4).

Floor beams on column center lines can be framed to the channels with the usual clip angle connection (Detail 5).





Channels-To-Wide-Flange Connection

Where channels and wide-flange are of the same depth, the connection requires a tee, an angle and a plate, with the sizes determined by the design moment and shear (Detail 6).

Where channels are deeper than the wide flange, a similar approach can be taken, using two plates and one tee (Detail 7).

Features Of The Straddle-Beam Tree Column

- Simple fabrication and erection procedures.
- Greater shop and less field work.
- Welding economy and easier inspection.
- Option to use all bolted field connections.
- Provides for beam continuity around the column rather than through it.
- Vertical piping and conduit can be installed adjacent to the column.

Robert L. Boehmig, P.E., is a recently retired engineer living in Atlanta.

Hot Dip Galvanizing

A primer on designing and fabricating structural steel to be hot dip galvanized after fabrication

By Philip G. Rahrig

CORROSION AND REPAIR OF CORROSION DAMAGE ARE MULTI-BILLION DOLLAR PROBLEMS, conservatively estimated to be $222 billion per year in the United States alone. Hot dip galvanizing after fabrication is a versatile corrosion control process which can help solve many of these problems in industrial, transportation, utility and building construction.

An excellent source of information on galvanizing is the American Galvanizers Association (AGA), together with its members. The expertise and experience of these galvanizers provides a baseline of "do's and don'ts" for designers. Another good source is the standardized set of recommended design and details of design that have been captured in two publications of the AGA: *The Design of Products To Be Hot Dip Galvanized After Fabrication* and *Recommended Details For Galvanized Structures*. Finally, ASTM has developed specifications to cover the effective design of structurals and structural assemblies to produce high quality galvanized products. All three sources of information are used to provide the following overview on successful product design. (More information on galvanizing will be provided in the June 1995 *Modern Steel Construction* in an article discussing cope cracking with galvanized steel.)

The Galvanizing Process

Galvanizing has proven to be a successful answer to corrosion problems because of the unique nature of this coating. As a barrier coating, it provides a tough, metallurgically bonded zinc coating which completely covers the steel surface and seals it from the corrosive action of its environment. In addition, the sacrificial action of zinc protects the steel even where damage or minor discontinuity occurs in the coating.

Galvanizing is a worldwide method of protection against corrosion. *As governments and industry become more aware of the cost of corrosion, galvanizing will play a more prominent role in protecting the integrity of our buildings, roads, bridges and other structures. For more information about galvanizing and the galvanizer nearest you, contact the American Galvanizers Association 1-800-468-7732. The following AGA publications provide a wealth of information to designers, specifiers and fabricators:*

- *Selected Specifications for Hot Dip Galvanizing Inspection of Products Hot dip Galvanized After Fabrication*
- *Galvanizing for Corrosion Protection—A Specifiers Guide*
- *Recommended Details of Galvanized Structures*
- *Painting Galvanized Steel*

In order for the designer and fabricator to visualize the possible effects that galvanizing may have on the fabricated article, it is necessary to understand the process of galvanizing. Though the process may vary slightly from galvanizer to galvanizer, the fundamental steps in the galvanizing process are:

1. Soil and Grease Removal. A hot alkaline cleaner is used to remove oil, grease, shop oil and soluble paints. This will not, however, remove such things as epoxies, vinyl, asphalt or welding slag. These surface contaminants must be removed by grit blasting, sandblasting or other mechanical cleaning methods and are typically the responsibility of the fabricator.

2. Pickling. A dilute acid solution is used to remove surface rust and mill scale to provide a chemically clean metallic surface.

3. Prefluxing. A steel article may be immersed in a liquid preflux solution (usually zinc ammonium chloride) to remove oxides and to prevent oxidation prior to immersion into the molten zinc.

4. Galvanizing. The article is immersed into a kettle of molten zinc, which may be covered with a surface layer of flux, at approximately 850 degrees Fahrenheit. This results in the formation of a zinc and zinc-iron alloy coating which is metallurgically bonded to the steel.

5. Finishing. After the article is withdrawn from the galvanizing bath, excess zinc is removed by draining, vibrating or, for small items, centrifuging. The galvanized items are then cooled in air or quenched in water.

6. Inspection. Thickness and surface condition inspections are the final steps in the process. Thickness of the zinc coating is determined by making magnetic thickness measurements.

Galvanizing Considerations

The most important rule when specifying galvanized steel is to involve the designer, fabricator and galvanizer before the product is designed. This three-way communication can eliminate most problems that may occur. In fact, it is a requirement of *ASTM A123—Standard Specifications for Zinc (Hot Dip Galvanized) Coatings on Iron and Steel Products.*

Within the processing parameters describe above, there are specific design and fabricating considerations to insure that the resultant hot dip galvanized coating meets all of the customer requirements for quality in appearance and maintenance-free corrosion protection while in service. In addition to existing specifications that cover fabrication considerations, there are also many important practical standards that should be heeded in order to obtain a high quality hot dip galvanized product.

The first and most important step for the designer is to establish a realistic design, clearly covered by existing specifications. Along with shop drawings, the fabricator should submit a copy of the specifications to which the galvanizer is expected to adhere. Secondly, the galvanizer should acknowledge receipt of the drawings and specifications and identify any areas of concern, based upon the vast experience of galvanizing articles of all sizes and shapes. This method of concurrent design should insure a high quality galvanized product.

Existing Specifications

The following specifications from the American Society of Testing Materials are important to insure a suitable fabrication to be galvanized. Their salient points are outlined below.

ASTM A-143

This specification describes the *Standard Practice for Safeguarding Against Embrittlement of Hot Dip Galvanized Structural Steel Products and the Procedure for Detecting Embrittlement.*

1. Embrittled galvanized steel is defined as steel that has lost its ductile properties. While it may be complete or partial, ebrittlement seldom occurs except at points which have been severely cold worked.

2. Selection of the proper steel to be fabricated and the design of the product to withstand normal galvanizing processes is the responsibility of the designer. The galvanizer is responsible for using the proper pickling duration and zinc bath temperature to prevent embrittlement damage to the product.

3. Factors Contributing to embrittlement:

- Any form of cold working reduces the ductility of rolled steel.
- Pickling roughens the surface and, therefore, has some effect on reducing its ductility. However, the major effect of pickling is an increase in hydrogen absorption by the steel, which may contribute to embrittlement.
- Galvanizing normally does not produce injurious loss of ductility unless the material has previously received severe localized cold working.
- Embrittlement is related to temperature. The lower the ambient temperature during fabrication, the greater the tendency of steel toward embrittlement. This tendency is increased by cold working and by notch effects.

4. If any cold working without subsequent annealing or stress relieving is to precede galvanizing, then only open-hearth basic oxygen and electric furnace steels should be used.

5. For intermediate and heavy shapes and plates, sharp bending should be performed hot. If steel is bent cold, annealing or stress relieving should follow.

ASTM A-384

This specification describes the Recommended Practice for Safeguarding Against Warpage and Distortion During the Hot Dip Galvanizing of Steel Assemblies.

1. All fabricated assemblies have a tendency to distort as a result of stresses induced during the manufacture of components and subsequent fabrication.

2. As a result of immersion into and withdrawal from the molten zinc bath at a temperature of 820 - 860 degrees F and the alternate heating and cooling incidental to the galvanizing process, warpage and distortion may occur.

The American Galvanizers Association GalvaTech Seminar Series is designed for design and specification professionals and provides a comprehensive overview on the "Application and Specification of Galvanizing for Corrossion Protection."

The seminar costs $50, and includes an introductory professional membership in AGA as well as a technical binder containing a compilation of AGA literature (a $45 value). For more information, call 800/788-4258.

1995 Schedule

May 9	Baltimore
May 11	Birmingham, AL
Oct. 3	Boston
Oct. 5	Orlando, FL
Oct. 17	New Orleans
Oct. 19	Chicago
Nov. 7	Portland, OR
Nov. 9	Denver

3. Factors in warpage and distortion:

- Assemblies of sheet or thin gauge plate welded or riveted to bars or structural section are two of the most commonly warped or distorted items.
- Warpage is almost assured when non-symmetrical sections such as channels are used. In nearly every instance, channels require straightening after galvanizing. Symmetrical sections should be used whenever possible.
- Wide radii bends should be used to minimize local stress concentrations in corners.

4. Suggestions for minimizing or avoiding warpage and distortion:

- The extent to which an assembly will distort is largely a function of design and can be significantly reduced by selecting the structural members with the minimum weight and cross section.
- All parts of a fabrication should be of equal thickness so that rates of expansion and contraction will be alike. Assemblies with sheet metal or expanded metal covers should be avoided.
- Sealed, continuous welds are preferred but if impractical, staggered welds to minimize induced stresses should be specified.
- Assemblies should be preformed accurately so that force is not required to bring them into position.
- Finite kettle lengths make double dipping of long assemblies necessary. If possible, design the assemblies to fit into the kettle in a single dip process. This eliminates uneven expansion and contraction that may occur when double dipping.
- Bracing and reinforcing are recommended to avoid warpage and distortion in some assemblies. Consult your galvanizer for recommendations. (See Fig 1)

ASTM A-385

This specification describes

the recommended practice for providing high quality hot dip zinc coatings on assembled products.

1. For overlapping or contacting surfaces: (See Fig. 2)

• All edges of tightly contacting surfaces should be completely sealed by welding. While pickling acids will penetrate into the unsealed areas, zinc metal will not. Moisture and corrosion products will bleed from the contacting but unwelded surfaces. Additionally, the possibility of explosion exists as the trapped cleaning liquids vaporize and expand at the zinc bath temperature of 850 degrees F. Danger of explosion is more acute for larger overlapped areas. According to the following guidelines, it is recommended that a vent be provided in one of the overlapping plates:

Overlapped Area (Sq. In.)	Venting Requirements
less than 16	none
more than 16 and less than 64 when steel is 1/2-in. thick	one 3/8-in. dia. hole, or leave 1-in. of unwelded area
more than 16 and less than 64 when steel is more than 1/2-in. thick	none
more than 64 and less than 400	one 1/2-in. dia. hole, or leave 2-in. of unwelded area
Each increment of 400	one 1/4-in. dia. hole, or leave 4-in. of unwelded area

• Where bars are to join at an angle and a 3/32" space is provided to allow acid to flow freely, a periodic fillet weld may be used on either or both sides. This type of welding is not recommended for loadbearing members.

2. For sheet steel rolled over wire or rod stiffener, all oil and grease should be cleaned off before rolling. Grease at the high galvanizing temperatures becomes volatile and generates a gas which prevents the molten zinc from sealing the contact edges.

3. When arc welding, all residue must be removed prior to pickling. Sand blasting, chipping, wire brushing or other descaling processes are appropriate. Because welding residue is chemically inert, it will not be cleaned in the galvanizing process and will eventually work its way under the residue during use. An unsightly and unprotected area is thus susceptible to corrosion.

4. Cast iron, cast steel and malleable iron assembled together with rolled steel should be sand blasted after assembly. This helps prevent a dissimilar galvanized finish that may results because of the different pickling characteristics of different materials. Similarly, different materials (surface finish and/or chemical composition) should be galvanized separately.

5. Pipe fabrications (See Fig. 3) must have sufficient openings in number, total area and proper location to permit interior gases and moisture to escape during the pickling and galvanizing process. If this precaution is not taken, there is a large potential for explosion caused by the buildup of trapped gas pressure at the very high galvanizing temperature.

• Pipe railing may be vented internally or externally. Internal venting is preferred with a minimum vent hole of 3/8" diameter.

• Pipe columns, similar to railing, should be vented through the top and bottom plates or through the walls of the pipe. The holes should be as large as possible and diagonally opposite at 180 degrees.

• Pipe should be ordered without mill lacquer if it is intended to be galvanized.

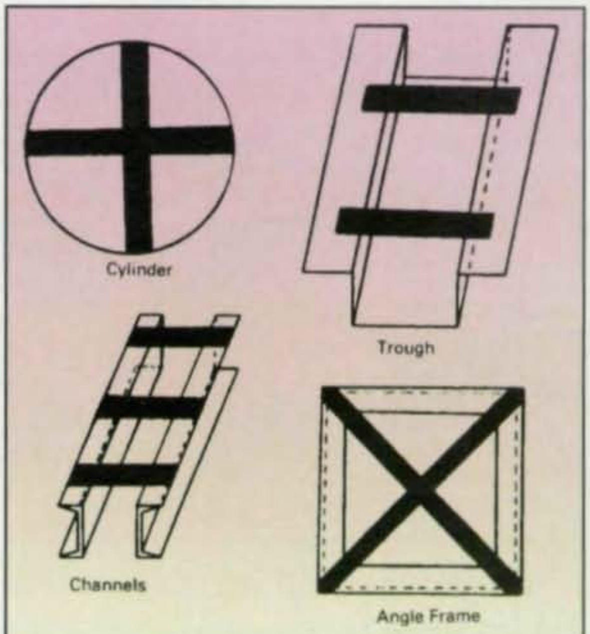


Figure 1

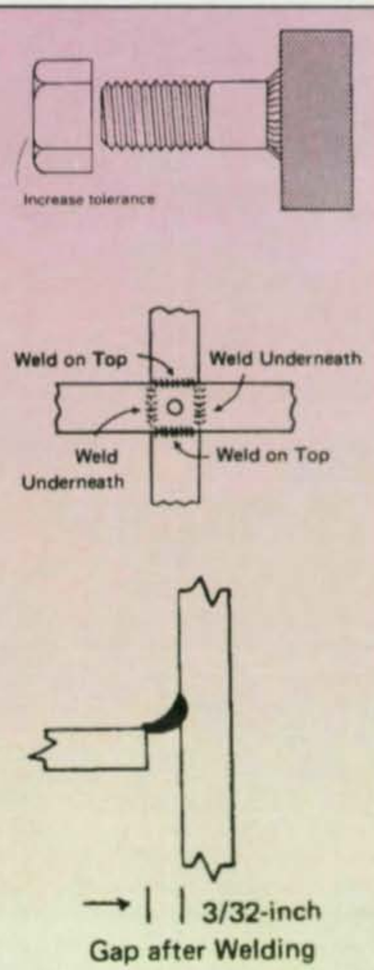


Figure 2

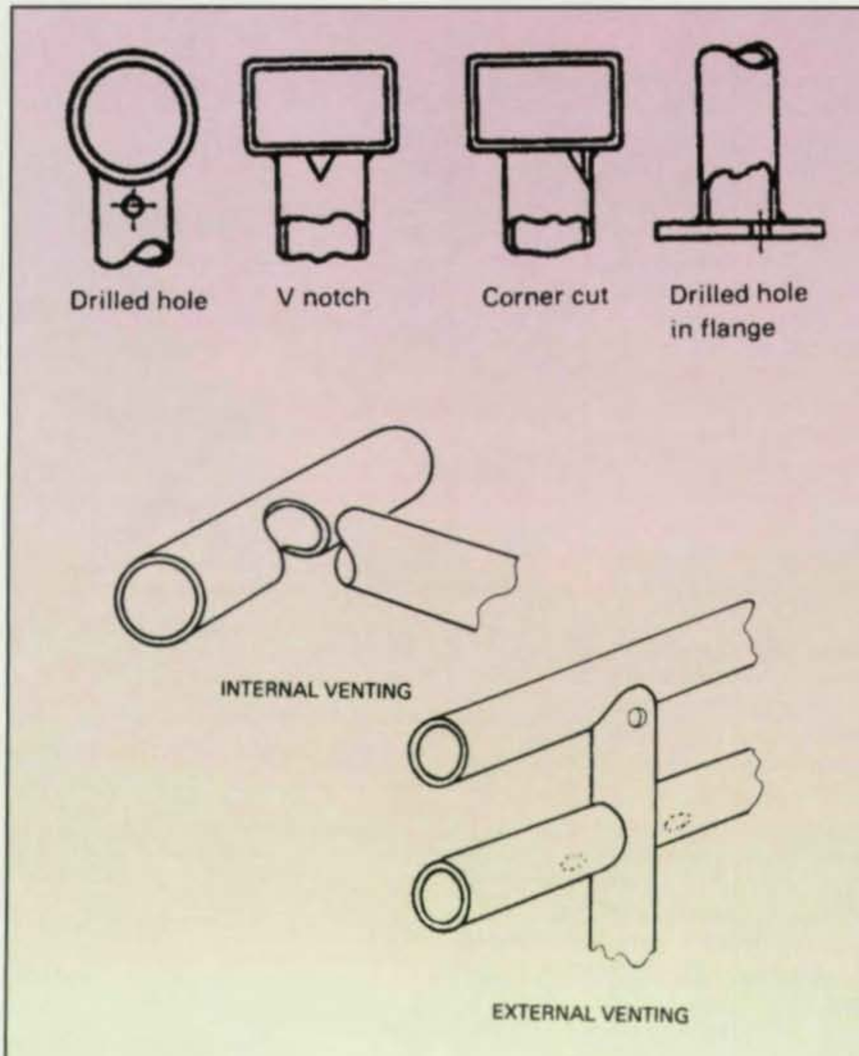


Figure 3

Lacquered pipe tends to carburize and burn about 6" from weld areas. Because zinc will not adhere to the carburized surface, the area should be sandblasted prior to pickling.

• Fittings should be uncoated and made of steel. If cast iron fittings must be used, they should be sandblasted prior to pickling.

6. Labels and markings for fabrication identification should be water soluble. If markings are absolutely necessary, deep stencil, weld bead or stainless steel stenciled tags should be firmly affixed. Aluminum, plastic, paper or paint markings will not be readable after galvanizing.

7. Galvanizing should not be considered to be a filler. It will not fill in pits in old or heavily rusted steel to provide a uniform surface.

8. For fasteners under 1-1/2" in size, the standard practice is for the bolt to be threaded standard and the nut tapped oversized after galvanizing. Although not a part of ASTM A384, for fasteners over 1-1/2", the opposite practice is employed. Although tapping and/or retapping after galvanizing results in an uncoated thread, the zinc coating on the opposite piece of fastener acts sacrificially to protect the uncoated surface.

Practical Considerations

1. The galvanizer should always be consulted when determining what type of steel should be used for a fabrication that is to be galvanized. A low carbon, low silicon, mild steel will usually yield the most uniform coatings but deviations can materially affect the necessary pickling techniques and the structure and appearance of the galvanized coating.

2. For the best drainage and run off of the pre-flux and molten zinc, avoid boxed-in sections whenever possible. Notch fabrications with stiffeners in order to permit the free flow of zinc to and away from all surfaces (See Fig. 4). Not taking this precaution results in unsightly buildup of zinc.

3. All bending, forming and punching should be done before galvanizing. If done after galvanizing, there is a high potential that the zinc coating will crack.

4. If appearance of the finished fabrication is critical, a lifting mechanism should be provided when the fabrication is delivered to the galvanizer. Normal operation calls for the fabrication to be immersed into the galvanizing kettle from overhead, using wire or chains to connect the fabrication with the hoisting device. In general, these wires and chains leave a mark on the fabrication, a mark that is not necessarily detrimental to the corrosion protection of the coating, but mars the surface appearance.

5. Preservatives should not be used on articles to be galvanized.

6. Normal pickling solutions will not attack foundry sand. Thus, all casting should be sandblasted prior to galvanizing. Finish machining should be done after sandblasting.

7. Sections with hinges, swivels or moving parts should be disassembled before galvanizing and reamed/cleaned afterward to remove zinc coating from moving parts.

Handling & Storage

Although galvanized steel is extremely durable in service life,

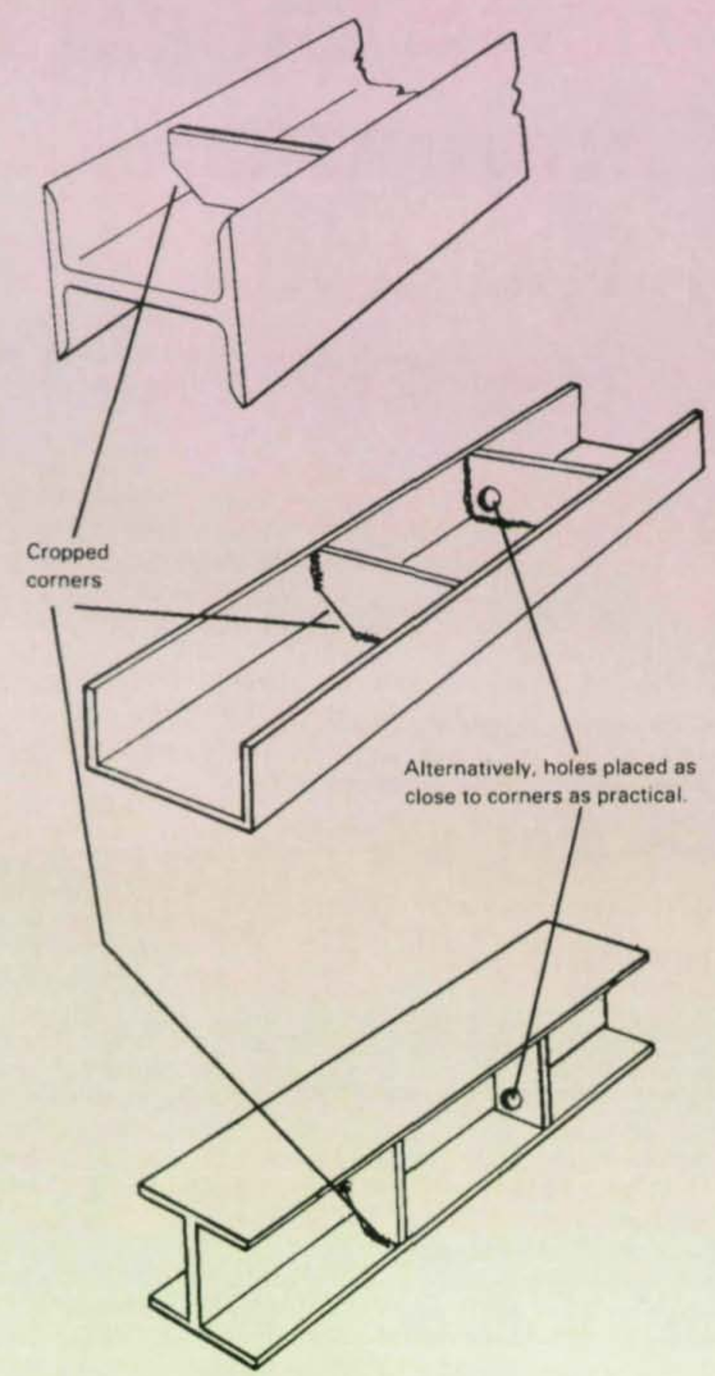


Figure 4

care should be taken to minimize the mechanical damage to the steel during transport, handling and storage. When storing newly galvanized articles outdoors, care should be taken to stack the steel so that moisture will not gather on or within any galvanized articles. Failure to do this could accelerate the sacrificial action of the zinc, resulting in an accelerated rate of coating corrosion.

Philip G. Rahrig is executive/marketing director with the American Galvanizers Association.

AUTOMATED STRUCTURAL DESIGN CALCULATIONS USING MATHCAD

By Thomas Magner

THE FORMULAS IN TODAY'S DESIGN CODES AND SPECIFICATIONS HAVE RENDERED MANUAL CALCULATIONS IMPRACTICAL for all but the simplest structural design problems. Mathcad 5.0 software (MathSoft, Inc., Cambridge, Massachusetts), which runs under Microsoft Windows on 386- or 486-based PCs and compatibles, is a unique way of working with formulas, numbers, text and graphs that combines the advantages of manual calculations with the power of computer calculations. Mathcad was developed to perform engineering and scientific calculations and uses familiar math notation in a free-form scratchpad format that is familiar to engineers.

Formulas in Mathcad are written in standard mathematical notation, the equations are solved automatically, and the answers are presented as single variables, arrays, tables, or graphs. Unlike a specialized computer or spreadsheet program program, everything in Mathcad is visible in a WYSIWYG environment for review or editing by the engineer. In contrast, PC-based spreadsheet programs have powerful features, but not all engineering problems fit conveniently into tabular format; the formulas in a spreadsheet are also "hidden" in the cells and it is more difficult to review and edit a spreadsheet template. Specialized programs also are difficult to review and change and require experience in programming.

Because of the familiar mathematics interface, Mathcad can be used by a beginner in place of manual calculations after a few days of familiarization with the program. Calculations are easily revised or corrected, and the entire document automatically updates. The document may then be saved for future reference or printed out exactly as it appears on screen.

The full potential of Mathcad is exploited by applying advanced features to repetitive calculations. This article describes a Mathcad document for the automated design of any laterally braced, unstiffened plate girder or built-up beam section using the *Load and Resistance Factor Design Specification for Structural Steel Buildings, Effective December 1, 1993*. A similar Mathcad document for design of sections using stiffeners could be prepared to examine alternate designs using stiffeners. Included with this article is an example of a complete working Mathcad document.

The application shown uses only a few of Mathcad's features: user-defined functions; the if function (which permits conditional calculations); the min, max, floor, ceil, and root functions; logical operators (greater than, less than, equal to, etc.); vectors and range variables; and the Mathcad solve block. Units are incorporated in the equations to permit use of Mathcad's unit conversion capability, since Mathcad builds in an extensive list of MKS, CGS and U.S. Customary units. The user can easily convert units or to define custom units in terms of the built-in ones. For example, this document may be converted from U.S. to MKS units in a few minutes.

> Because of its familiar mathematics interface, the program is fast to learn and use

DESCRIPTION, INPUT VARIABLES AND CONSTANTS

A description of the document, input variables, and constants are shown in the example document. The user can specify values for moment, shear, yield strengths, nominal section depth, and maximum and minimum flange width.

RdF values (shown as a vector) are dimensional multiples used to round theoretical plate dimensions to practical dimensions. Rounding multiples may also be specified by the user. Individual RdF values are referred to by using their vector index numbers as subscripts: 0 for flange width, 1 for flange width, 2 for web thickness, and 3 for web depth. The input values shown in this document may be easily changed by the user to design any unstiffened plate girder or built-up beam. Mathcad's assignment operator (a colon followed

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73	74	75	76	77	78	79	80	81	82	83	84
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by an equal sign) assigns values to input variables. The min function is used to select the smaller limiting value for the ratio of web height to web thickness for a plate girder. This built-in function takes the form min(A), where A is a vector or matrix, and returns the minimum value in the array.

FUNCTIONS

Since the variables flange width, flange thickness, section depth, web thickness, and ratio of web height to web thickness are unknown, all other variables are expressed as user-defined functions of these values. User-defined functions in Mathcad, like built-in functions, are expressed in standard mathematics notation, with the variable name followed by the "arguments" within brackets. Because of the live mathematics interface, multiplication by is expressed using the Mathcad multiplication operator, an elevated dot. Once a function is defined in a document, it may be evaluated for particular values of the arguments at any point in the document below or to the right.

All necessary functions are defined in the middle part of the example document. Where possible, the function names use the same notation as that used in the LRFD Specification. Logical operators (e.g., equal to, greater than or equal to, etc.), the if, max, and root built-in functions, and a Mathcad solve block are used in defining functions.

The Mathcad if function is used in the defining the hybrid girder factor and the plate girder strength reduction factor, as well as the nominal flexural and shear strengths. For example, depending on the ratio of web height to thickness, the nominal flexural strength is equal to the plastic moment, the moment capacity limited by web buckling for a noncompact beam section, or the moment capacity for a plate girder. The syntax of the if function is if(cond,tval,fval). The condition cond usually involves one or more expressions that use logical operators. The value tval is returned if the condition is true, and fval is returned if the condition is false. if functions may also be nested, as shown in the definition of nominal flexural strength.

Both the if and max functions are usedto define the hybrid girder factor. The hybrid girder factor is 1.0 if the flange and web strengths are equal, or it is equal to the larger of the value computed by the formula within the two-element vector or 1.0. The max function is similar to the min function, and returns the maximum value in an array.

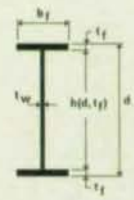
The max and root functions are used in defining the function R for calculating the required web thickness for any given flange thickness. The R function defines the web thickness as the larger of the value required to satisfy the limiting web ratio or the thickness required for the maximum shear. The Mathcad root function takes the form root(f(x),x) and returns the value of x that makes the function f equal to 0.

Example Document

DESCRIPTION

The calculations in this document determine the required plate dimensions for a laterally braced, unstiffened plate girder or built-up welded beam section following the requirements of the **Load and Resistance Factor Design Specification for Structural Steel Buildings**. Effective December 1, 1993. American Institute of Steel Construction.

USER INPUT



Maximum bending moment:	$M_u := 14400 \cdot kip \cdot ft$
Maximum vertical shear:	$V_u := 570 \cdot kips$
Flange yield strength:	$F_{yf} := 50 \cdot ksi$
Web yield strength:	$F_{yw} := 36 \cdot ksi$
Minimum flange width:	$b_{min} := 8 \cdot in$
Maximum flange width:	$b_{max} := 24 \cdot in$
Nominal section depth:	$d := 72 \cdot in$

Constants

Resistance factor for flexure: $\phi_b := 0.9$ Resistance factor for shear: $\phi_v := 0.9$

Compressive residual stress for welded shapes: $F_r := 16.5 \cdot ksi$ Ratio of web yield stress to flange yield stress: $m := \frac{F_{yw}}{F_{yf}}$ $m = 0.72$

Flange width multiple:
Flange thickness multiple:
Web thickness multiple:
Web depth multiple:

$$RdF := \begin{pmatrix} 2 \\ 0.125 \\ 0.0625 \\ 2 \end{pmatrix} \cdot in$$

Plate dimensions are rounded to multiples of the RdF factors. The final section depth may be greater or less than the nominal depth due to rounding of web height and flange thickness.

Limiting ratios of height to thickness for webs of compact beams, noncompact beams, and plate girders (LRFD Spec. Table B5.1 and Appendix Section G1)

Compact: $WR_c := 640 \cdot \sqrt{\frac{ksi}{F_{yf}}}$ $WR_c = 90.51$

Noncompact: $WR_{nc} := 970 \cdot \sqrt{\frac{ksi}{F_{yf}}}$ $WR_{nc} = 137.179$

Plate girder: $WR_{pg} := \min\left(\begin{bmatrix} 260 \\ \dfrac{14000}{\sqrt{\frac{F_{yf}}{ksi} \cdot \frac{(F_{yf} + F_r)}{ksi}}} \end{bmatrix}\right)$ $WR_{pg} = 242.791$

Minimum compact flange thicknesses corresponding to flange widths b_{min} and b_{max} (LRFD Spec. Table B5.1)

$t_{f1} := \frac{b_{min}}{130} \cdot \sqrt{\frac{F_{yf}}{ksi}}$ $t_{f1} = 0.435 \cdot in$ $t_{f2} := \frac{b_{max}}{130} \cdot \sqrt{\frac{F_{yf}}{ksi}}$ $t_{f2} = 1.305 \cdot in$

FUNCTIONS

Independent Variables

Flange width: b_f Web thickness: t_w Ratio of web height to web thickness: WR

Flange thickness: t_f Section depth: d

Web height: $h(d, t_f) := d - 2 \cdot t_f$

Section area: $A(b_f, d, t_f, t_w) := (2 \cdot b_f \cdot t_f + h(d, t_f) \cdot t_w)$

Girder weight: $Wt(b_f, d, t_f, t_w) := A(b_f, d, t_f, t_w) \cdot 3.4 \cdot \frac{plf}{in^2}$

Flange plastic section modulus: $Z_f(b_f, d, t_f) := b_f \cdot t_f \cdot (h(d, t_f) + t_f)$

Web plastic section modulus: $Z_w(d, t_f, t_w) := \frac{t_w \cdot h(d, t_f)^2}{4}$

Plastic section modulus: $Z(b_f, d, t_f, t_w) := Z_f(b_f, d, t_f) + Z_w(d, t_f, t_w)$

Plastic moment: $M_p(b_f, d, t_f, t_w) := Z_f(b_f, d, t_f) \cdot F_{yf} + Z_w(d, t_f, t_w) \cdot F_{yw}$

Moment of inertia: $I_x(b_f, d, t_f, t_w) := \frac{b_f \cdot d^3 - (b_f - t_w) \cdot (d - 2 \cdot t_f)^3}{12}$

Elastic section modulus: $S_x(b_f, d, t_f, t_w) := \frac{2 \cdot I_x(b_f, d, t_f, t_w)}{d}$

Ratio of web area to compression flange area: $a_r(b_f, d, t_f, t_w) := \frac{h(d, t_f) \cdot t_w}{b_f \cdot t_f}$

Hybrid girder factor: (LRFD Spec. App. G2)

$$R_e(b_f, d, t_f, t_w) := \mathrm{if}\left[F_{yf} = F_{yw}, 1.0, \max\left(\begin{bmatrix} \dfrac{12 + a_r(b_f, d, t_f, t_w) \cdot (3 \cdot m - m^3)}{12 + 2 \cdot a_r(b_f, d, t_f, t_w)} \\ 1.0 \end{bmatrix}\right)\right]$$

Plate girder bending strength reduction factor: (LRFD Spec. Eq. (A-G2-3))

$$R_{PG}(b_f, d, t_f, t_w) := \mathrm{if}\left(\frac{h(d, t_f)}{t_w} \le \frac{970}{\sqrt{F_{yf}}} \cdot \sqrt{ksi}, 1, 1 - \frac{a_r(b_f, d, t_f, t_w)}{1200 + 300 \cdot a_r(b_f, d, t_f, t_w)} \cdot \left(\frac{h(d, t_f)}{t_w} - \frac{970}{\sqrt{F_{yf}}} \cdot \sqrt{ksi}\right), 1\right)$$

Later, an if function is used to define the function G(WR) which provides the guess values for flange width, flange thickness, and web thickness for use in applying the solve block function which follows.

In this application a Mathcad solve block, defined as a function, is used to eliminate the usual iterative method for selecting a section to meet specified criteria. The Mathcad solve block construct can solve up to 50 simultaneous equations by successive approximation; the equations may be linear, nonlinear, or transcendental. The Mathcad solve block structure must include guess values for the unknowns, the keyword Given, equations involving the unknowns, and a call to the Find function with a list of the unknowns. Guess values provide the solve block with a starting point to begin the solution.

> The Mathcad solve block construct can solve up to 50 simultaneous equations

By using guess values within the limits of the equations in the solve block, and avoiding equations which are discontinuous, the solver consistently finds the required answers over the entire range of practical input values. The solve block is sensitive to the initial guess values and requires different guess values for plate girders, noncompact sections, and compact sections. Different guess values for each section type are also necessary depending on whether the final flange width will be between the minimum and maximum flange widths or will be equal to the maximum flange width. The function G(WR) in the example document provides the guess values of flange width, flange thickness, and web thickness for each type of section.

The first equation within the solve block shown in middle of the example document requires the flexural capacity to be equal to the moment. The second equation requires the web thickness to be equal to the larger of the thicknesses required for shear or to meet the specified limiting web ratio. The third equation requires the flange width to be equal to the smaller of the maximum flange width or the

Example Document, Continued

Nominal flexural strength of a plate girder with a compact flange:
(LRFD Spec. Eq. (A-G2-2))

$$M_G(b_f,d,t_f,t_w) := S_x(b_f,d,t_f,t_w)\cdot R_{PG}(b_f,d,t_f,t_w)\cdot R_e(b_f,d,t_f,t_w)\cdot F_{yf}$$

Limiting buckling moment for a beam with a noncompact web:
(LRFD Spec. App. Table A-F1.1)

$$M_r(b_f,d,t_f,t_w) := R_e(b_f,d,t_f,t_w)\cdot F_{yf}\cdot S_x(b_f,d,t_f,t_w)$$

Nominal flexural strength for a noncompact beam (limit state of web local buckling):
(LRFD Spec. Eq. (A-F1-3))

$$M_W(b_f,d,t_f,t_w) := M_p(b_f,d,t_f,t_w) - \left(M_p(b_f,d,t_f,t_w) - M_r(b_f,d,t_f,t_w)\right)\cdot\left(\frac{h(d,t_f)}{330\cdot t_w} - \frac{\sqrt{\frac{F_{yf}}{ksi}}}{330}\cdot 640\right)$$

Nominal flexural strength for compact beam, beam with a noncompact web, or plate girder:

$$f1(b_f,d,t_f,t_w) := \mathrm{if}\left(\frac{h(d,t_f)}{t_w} \le \frac{970}{\sqrt{F_{yf}}}\cdot\sqrt{ksi},\ M_W(b_f,d,t_f,t_w),\ M_G(b_f,d,t_f,t_w)\right)$$

$$M_n(b_f,d,t_f,t_w) := \mathrm{if}\left(\frac{h(d,t_f)}{t_w} \le \frac{640}{\sqrt{F_{yf}}}\cdot\sqrt{ksi},\ M_p(b_f,d,t_f,t_w),\ f1(b_f,d,t_f,t_w)\right)$$

Nominal shear strength
LRFD Spec. Eq. (F2-1): $V_{n1}(d,t_w) := 0.6\cdot F_{yw}\cdot d\cdot t_w$

Nominal shear strength
LRFD Spec. Eq. (F2-2):

$$V_{n2}(d,t_f,t_w) := 0.6\cdot F_{yw}\cdot d\cdot t_w\cdot\left(\frac{418\cdot t_w\cdot\sqrt{ksi}}{\sqrt{F_{yw}}\cdot h(d,t_f)}\right)$$

Nominal shear strength
LRFD Spec. Eq. (F2-3):

$$V_{n3}(d,t_f,t_w) := \frac{132000\cdot d\cdot t_w^{3}}{h(d,t_f)^{2}}\cdot ksi$$

Nominal shear strength:
(LRFD Spec. Eq. (F2-1), Eq. (F2-2) or Eq. (F2-3))

$$f2(d,t_f,t_w) := \mathrm{if}\left(\frac{h(d,t_f)}{t_w} \le \left(\frac{523}{\sqrt{F_{yw}}}\cdot\sqrt{ksi}\right),\ V_{n2}(d,t_f,t_w),\ V_{n3}(d,t_f,t_w)\right)$$

$$V_n(d,t_f,t_w) := \mathrm{if}\left(\frac{h(d,t_f)}{t_w} \le \left(\frac{418}{\sqrt{F_{yw}}}\cdot\sqrt{ksi}\right),\ V_{n1}(d,t_w),\ f2(d,t_f,t_w)\right)$$

Function to determine web thickness with any flange thickness:

$$R(WR,t_f,t_w) := \max\begin{pmatrix}\dfrac{h(d,t_f)}{WR}\\ \mathrm{root}\left(\phi_v\cdot V_n(d,t_f,t_w) - V_u,\ t_w\right)\end{pmatrix}$$

Web thickness with minimum permissible flange size and specified maximum web ratio:
$t_{w1}(WR) := R(WR,t_{f1},0.5\cdot in)$

Shear capacity with minimum flange size and corresponding web thickness for specified maximum web ratio:

$V_{min}(WR) := \phi_v\cdot V_n(d,t_{f1},t_{w1}(WR))$

Moment capacity with minimum flange size and corresponding web thickness for specified maximum web ratio:

$M_{min}(WR) := \phi_b\cdot M_n(b_{min},d,t_{f1},t_{w1}(WR))$

Web thickness for specified maximum web ratio, for section with maximum flange width and corresponding minimum flange thickness:

$t_{w2}(WR) := R(WR,t_{f2},0.5\cdot in)$

Minimum moment for specified maximum web ratio, with maximum flange width and corresponding minimum flange thickness:

$M_{min2}(WR) := \phi_b\cdot M_n(b_{max},d,t_{f2},t_{w2}(WR))$

Guess values for flange width, flange thickness and web thickness:

$$G(WR) := \mathrm{if}\left(M_u < M_{min2}(WR),\ \begin{pmatrix}b_{min}\\ t_{f1}\\ t_{w1}(WR)\end{pmatrix},\ \begin{pmatrix}b_{max}\\ t_{f2}\\ t_{w2}(WR)\end{pmatrix}\right)$$

Solve block to determine theoretical section dimensions:

Given $M_u = \phi_b\cdot M_n(b_f,d,t_f,t_w)$

$$t_w = \max\begin{pmatrix}\dfrac{h(d,t_f)}{WR}\\ V_u\cdot t_w\\ \phi_v\cdot V_n(d,t_f,t_w)\end{pmatrix} \qquad b_f = \min\begin{pmatrix}b_{max}\\ \dfrac{130}{\sqrt{\frac{F_{yf}}{ksi}}}\cdot t_f\end{pmatrix} \qquad \begin{matrix}b_f \ge b_{min}\\ t_f \le \dfrac{d}{2}\end{matrix}$$

$S(WR,b_f,t_f,t_w) := \mathrm{Find}(b_f,t_f,t_w)$

CALCULATIONS

Guess values for flange width, flange thickness and web thickness, with web ratio limited to the maximum for a plate girder:

$$\begin{pmatrix}b_f\\ t_f\\ t_w\end{pmatrix} := G(WR_{pg}) \qquad \begin{pmatrix}b_f\\ t_f\\ t_w\end{pmatrix} = \begin{pmatrix}24\\ 1.305\\ 0.685\end{pmatrix}\cdot in \qquad WR := \frac{h(d,t_f)}{t_w} \qquad WR = 101.357$$

Number of possible section types (i.e. compact, noncompact or plate girder):

$N := \mathrm{if}(WR > WR_{nc},3,\mathrm{if}(WR > WR_c,2,1)) \qquad N = 2$

Theoretical section dimensions for a plate girder (0 dimensions if not feasible):

$$X^{\langle 0\rangle} := \mathrm{if}\left[N < 3,\ \begin{pmatrix}0\\0\\0\end{pmatrix}\cdot ft,\ \mathrm{if}\left(M_u \le M_{min}(WR_{pg}),\ G(WR_{pg}),\ S(WR,b_f,t_f,t_w)\right)\cdot ft\right] \qquad X^{\langle 0\rangle} = \begin{pmatrix}0\\0\\0\end{pmatrix}\cdot in$$

Guess values for flange width, flange thickness, web thickness and web ratio for a noncompact beam:

$$\begin{bmatrix}b_f\\ t_f\\ t_w\end{bmatrix} := \mathrm{if}\left[N = 3,\ G(WR_{nc}),\ \begin{pmatrix}b_f\\ t_f\\ t_w\end{pmatrix}\right] \qquad \begin{pmatrix}b_f\\ t_f\\ t_w\end{pmatrix} = \begin{pmatrix}24\\ 1.305\\ 0.685\end{pmatrix}\cdot in \qquad WR := \frac{h(d,t_f)}{t_w} \qquad WR = 101.357$$

Theoretical section dimensions for a beam with a noncompact web (0 dimensions if not feasible):

$$X^{\langle 1\rangle} := \mathrm{if}\left[N < 2,\ \begin{pmatrix}0\\0\\0\end{pmatrix}\cdot ft,\ \mathrm{if}\left(M_u \le M_{min}(WR_{nc}),\ G(WR_{nc}),\ S(WR,b_f,t_f,t_w)\right)\cdot ft\right] \qquad X^{\langle 1\rangle} = \begin{pmatrix}24\\ 1.97\\ 0.676\end{pmatrix}\cdot in$$

Guess values for flange width, flange thickness, web thickness and web ratio for a compact beam:

$$\begin{bmatrix}b_f\\ t_f\\ t_w\end{bmatrix} := \mathrm{if}\left[N = 2,\ G(WR_c),\ \begin{pmatrix}b_f\\ t_f\\ t_w\end{pmatrix}\right] \qquad \begin{pmatrix}b_f\\ t_f\\ t_w\end{pmatrix} = \begin{pmatrix}24\\ 1.305\\ 0.767\end{pmatrix}\cdot in \qquad WR := \frac{h(d,t_f)}{t_w} \qquad WR = 90.51$$

Theoretical section dimensions for a compact section:

$$X^{\langle 2\rangle} := \mathrm{if}\left(M_u \le M_{min}(WR_c),\ G(WR_c),\ S(WR,b_f,t_f,t_w)\right)\cdot ft \qquad X^{\langle 2\rangle} = \begin{pmatrix}24\\ 1.908\\ 0.753\end{pmatrix}\cdot in$$

Theoretical section weights:

$i := 0..2 \quad j := 0..2 \qquad W_j := Wt(X_{0,j},d,X_{1,j},X_{2,j}) \qquad W^T = (0\ \ 478\ \ 486)\cdot plf$

Theoretical section weights with zero weights defined equal to the maximum weight plus 1 plf:

$W'_i := \mathrm{if}(W_i = 0\cdot plf,\ \max(W) + plf,\ W_i) \qquad W'^T = (487\ \ 478\ \ 486)\cdot plf$

Index # of lightest section (greater than 0 plf):

$U := \mathrm{if}\left[(W'_0 = \min(W')),\ 0,\ \mathrm{if}(W'_1 = \min(W'),1,2)\right] \qquad U = 1$

Theoretical dimensions of the lightest section (greater than 0 plf):

$$\begin{pmatrix}b'_f\\ t'_f\\ t'_w\end{pmatrix} := X^{\langle U\rangle} \qquad \begin{pmatrix}b'_f\\ t'_f\\ t'_w\end{pmatrix} = \begin{pmatrix}24\\ 1.9703\\ 0.6758\end{pmatrix}\cdot in$$

Flange width and web depth rounded off to lower multiples:

$$b_f := \mathrm{floor}\left(\frac{b'_f + 0.01\cdot in}{RdF_0}\right)\cdot RdF_0 \quad b_f = 24\cdot in \qquad h_w := \mathrm{floor}\left(\frac{h(d,t'_f) + 0.01\cdot in}{RdF_3}\right)\cdot RdF_3 \quad h_w = 68\cdot in$$

Theoretical flange thickness revised if flange width is reduced:

$$t'_f := \frac{b'_f\cdot t'_f}{b_f} \qquad t'_f = 1.9703\cdot in$$

Flange thicknesses rounded up to next higher multiple:

$$t_f := \mathrm{ceil}\left(\frac{t'_f - 0.001\cdot in}{RdF_1}\right)\cdot RdF_1 \qquad t_f = 2\cdot in$$

Section depth revised if required due to changes in web height and flange thickness:

$d := h_w + 2\cdot t_f \qquad d = 72\cdot in$

Maximum permissible ratio of web height to thickness for lightest section:

$WR := \mathrm{if}(U = 0,\ WR_{pg},\ \mathrm{if}(U = 1,\ WR_{nc},\ WR_c)) \qquad WR = 137.179$

Theoretical web thickness revised if required due to changes in web height and flange thickness:

$t'_w := R(WR,t_f,t'_w) \qquad t'_w = 0.6754\cdot in$

Web thickness rounded to next higher multiple:

$$t_w := \mathrm{ceil}\left(\frac{t'_w - 0.001\cdot in}{RdF_2}\right)\cdot RdF_2 \qquad t_w = 0.6875\cdot in$$

Theoretical flange thickness revised if required due to changes in flange width, section depth and web thickness:

$$t'_f := \mathrm{if}\left(M_u > M_{min}(WR),\ \mathrm{root}\left(\phi_b\cdot M_n(b_f,d,t_f,t_w) - M_u,\ t_f\right),\ t'_f\right) \qquad t'_f = 1.961\cdot in$$

maximum compact flange width permitted by *LRFD Specification Section B5*. The two remaining inequalities prevent the solver from selecting a flange thickness greater than half the nominal depth or a flange width less than the minimum specified.

Calculations

The Mathcad document up to this point has allowed the user to specify critical input parameters and has defined functions and constraints for computing the solution. All that remains is to let Mathcad compute the results, as shown in the last third of the example document. The required theoretical dimensions for a plate girder, a beam section with a noncompact web, and a beam section with a compact web are computed.

If a section type is not feasible for the input values, 0-value dimensions for flange width, flange thickness, and web thickness are returned. The lightest section type is determined, the flange width and web depth are rounded off, and the flange thickness, web thickness, and section depth are rounded up.

The floor and ceil functions and the rounding multiples from earlier in the document are used to round plate dimensions to practical values. The ceil function takes the form ceil(x), and returns the smallest integer that is greater than or equal to x. Correspondingly, floor(x) returns the largest integer less than or equal to x. Small values are added or subtracted to the values within the brackets to prevent rounding the number to a higher value than necessary. Lastly, the computed dimensions and properties of the selected section are displayed, and the "custom" units kip, kips, and ksi are created from Mathcad's built-in unit set using Mathcad's global assignment operator. Since global definitions are interpreted before "local" definitions with the colon-equals assignment operator, these defined units may be placed anywhere in the document.

On a typical PC, the calculation of the document would take seconds. If a user substituted different input values in this document, the computations would update appropriately and automatically.

Conclusions

The Mathcad program provides a powerful yet flexible method of solving a wide range of structural design problems in an environment designed for engineering and scientific calculations. The Mathcad program offers advantages and features not available in either spreadsheets or specialized programs. The intuitive interface is easy to use, and can be used by a beginner to replace manual

Example Document, Cont.

Flange thicknesses rounded up to next higher multiple:

$$t_f := \mathrm{ceil}\left(\frac{t_f - 0.001\cdot in}{RdF_1}\right)\cdot RdF_1 \qquad t_f = 2\cdot in$$

Section depth revised if required due to changes in flange thickness:

$$d := h_w + 2 t_f \qquad d = 72\cdot in$$

Section Dimensions & Properties

Weight: $Wt(b_f, d, t_f, t_w) = 485\cdot plf$ Moment of inertia: $I_x(b_f, d, t_f, t_w) = 135646\cdot in^4$

Area: $A(b_f, d, t_f, t_w) = 142.7\cdot in^2$ Section modulus: $S_x(b_f, d, t_f, t_w) = 3768\cdot in^3$

Plastic section modulus: $Z(b_f, d, t_f, t_w) = 4155\cdot in^3$

Depth:
Flange width:
Flange thickness:
Web depth:
Web thickness:

$$\begin{bmatrix} d \\ b_f \\ t_f \\ h_w \\ t_w \end{bmatrix} = \begin{bmatrix} 72 \\ 24 \\ 2 \\ 68 \\ 0.6875 \end{bmatrix}\cdot in$$

Plate girder bending strength reduction factor: $R_{PG}(b_f, d, t_f, t_w) = 1$

Hybrid girder factor: $R_e(b_f, d, t_f, t_w) = 1$

Web ratio: $\frac{h_w}{t_w} = 98.9$

Limiting compact web ratio: $WR_c = 90.5$

Limiting noncompact web ratio: $WR_{nc} = 137.2$

Limiting plate girder web ratio: $WR_{pg} = 242.8$

Flexural strength: $\phi_b \cdot M_n(b_f, d, t_f, t_w) = 14635\cdot kip\cdot ft$

Shear strength: $\phi_v \cdot V_n(d, t_f, t_w) = 601.1\cdot kip$

Defined Units $kip \equiv 1000\cdot lbf$ $kips \equiv 1000\cdot lbf$ $ksi \equiv 1000\cdot \frac{lbf}{in^2}$ $plf \equiv \frac{lb}{ft}$

calculations after a short period of familiarization with the program. Since all equations in Mathcad are visible, errors can be corrected or changes can be made in variables or formulas within seconds.

Once a design problem is prepared in template form, as in the example shown in this article, the calculations do not have to be repeated for any problem that falls within the parameters of the template. If range variables are used in a template, all similar problems for a project can be completed in a single run of the application. Customized design tables or graphs can be prepared to a higher degree of accuracy then available in any design handbook — with the added advantage that the tables or graphs are based on a "live" mathematics interface, and can be changed by the user as needed.

Thomas Magner is a consulting engineer in New York and the author of the Mathcad Electronic Book Building Structural Design: Reinforced Concrete and Structural Steel Applications.

RECEIVE INFORMATION ON FABRICATION/DETAILING SOFTWARE THREE WAYS

1. Call an individual software vendor.

2. Circle an individual number on the Reader Service Card near the back of this magazine.

3. Circle #110 on the Reader Service Card and receive information on all the companies listed in this section.

AISC

AISC offers several programs for detailers and fabricators, including: AISC Database, which contains the properties and dimensions of structural steel shapes; CONXPRT, which is designed to produce fast, accurate connection designs; and AISC for AutoCAD, which produces shapes within AutoCAD.

For more information, contact AISC at 312/670-5411 or Circle #1.

AutoSD

AutoSD Steel Detailing offers more than 200 different LISP programs for quickly detailing steel designs. Also, a beam and column program allows the user to configure to individual standards while keeping different configurations on file. Blocks, cuts, minus dimensions and bolts are calculated from a database of AISC shapes. Bolts can be extracted and summarized using the separate Calculator program. The software works with AutoCAD R10, 11 or 12.

For more information, contact Ray Hoyt at 601/693-4729 or Circle #59.

CadVantage

CadVantage Structural Version 5.5 is a completely automated, stand-alone detailing system that combines the flexibility of individual input with the speed of batch processing. The company has introduced a Windows graphical interface system that runs on pen computers.

For more information, contact CadVantage at 704/344-9644 or Circle #45.

Computer Detailing Corporation

Computer Detailing offers three programs. Plans & Elevations is a menu-driven program that works inside of AutoCAD. Various scales can be used in the same drawing, without any calculations or conversions and structural shapes and elements can be automatically drawn to scale or exaggerated. The company's Beams & Columns program creates details, while the Optimal Cutting program finds the most economical nesting of material from either inventory, warehouse stock or a combination of the two.

For more information, contact Computer Detailing at 215/355-6003.

Computer Detailing Systems

This structural steel detailing software produces AutoCAD compatible drawings of shop details. Drawings are complete with dimensions, bill of materials and welds shown. Also produces engineering calculations, CNC data, and production control data. The program can interface with engineering programs through a steel detailing neutral file.

For more information, contact CDS at 803/552-7055 or Circle #63.

Design Data

SDS/2 from Design Data includes a variety of modules: detailing; engineering, analysis and design; estimating; and production control. The detailing module uses a solids modeling approach, which creates an exact replica of what is erected in the field. Automatic by-products of the model include details, connection design calculations, job status information, CNC data, and a bill or material report. The modules are linked, so information can be shared between modules.

For more information, contact Design Data at 800/443-0782 or Circle #32.

Dogwood Technologies

Metric output on drawing and material bill printouts is now available with PDS (Procedural Detailing System) from Dogwood Technologies. In addition, PDS also offers new fabrication downloads to the latest version of Peddimat, as well as to an ESAB burning table and a SEYCO drill line. Peddinghaus machines supported by the new programs are the APS angle machine, the TDK drill line and the ABC coping machine.

For more information, contact Dogwood Technologies at 800/467-0096 or Circle #93.

E.J.E. Industries

Several modules are now available for enhanced estimating and production. The Structural Material Manager accepts entries for material cost, shop hours and field hours and produces a job summary based on this information, while the Production Control Module prints shipping tickets and automatically records the date and quantity shipped. The Plate-Nesting Module finds the optimum cut of square and rectangular items from stock plates.

For more information, contact E.J.E. Industries at 800/321-3955 or Circle #46.

Fab/Trol Systems

Windows-compatible fabrication control software is now available from Fab/Trol Systems. Fab/Trol V5 is an integrated suite of programs designed to improve the management efficiency of metal fabrication shops and includes estimating, purchasing, nesting, inventory and remnant control, production control and shipping capabilities.

For more information, contact Fab/Trol systems at 503/485-4719 or Circle #94.

JJK AND ASSOCIATES

Quick-Connect from JJK and Associates simplifies connection design. Calculations are based on the ASD 9th Edition and feature: double angle bolted connections; bolted seated beam connections; bolted stiffened seated beam connections; bolted end plate framing connections; double angle welded framed beam connections; welded seated beam connections; beam splice plate connections; beam splice plate moment connections; end plate moment connections; bolted flange plate moment connections; beam flange directly welded to column moment connections; and welded flange plate moment connections.

For more information, call JJK and Associates at 505/275-8299 or Circle #95.

NES

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

For more information, contact NES at 800/637-1677 or Circle #96.

OMNITECH

Software program designs and details a wide variety of steel connections. A free demonstration disk is available.

For more information, contact Omnitech at 510/658-8328 or Circle #52.

ROMAC COMPUTER SERVICES

Written specifically for the steel fabrication industry, this software includes: material management; shop cutting lists; production tracking; shipping tickets; and other fabrication shop management tools. The Fabrication package can be purchased either as individual modules or as an integrated system.

For more information, contact Romac at 615/426-9634 or Circle #97.

STEEL 2000/STRUCAD 2000

Steel Solutions has coupled its production control program, STEEL 2000, with a detailing program, STRUCAD. The result, STRUCAD 2000, is a seamless integration of all aspects of estimating, detailing, multing, purchasing, receiving, production, shipping and erection with complete traceability. The software automatically creates the following from a 3D model: field erection drawings; shop detail drawings; fitting details, full-size templates; 3D erection drawings; and CNC manufacturing data for a wide range of workshop machinery. The software runs within Windows NT on Intel 486 or Pentium-based hardware, and interfaces with a variety of software packages, including AutoCAD; STAAD-III from Research Engineers; Intergraph's Mica Plus Analysis and PDS; PDMS from Cadcentre; and PASCE from CE Automation.

For more information, contact Ron Taylor at 304/472-2668 or Circle #98.

SOFTDESK STEEL DETAILER

Version 12 of Softdesk's Steel Detailer module provides the tools to create shop fabrication drawings. The program couple structural information and parametric programs to quickly produce accurate and high-quality drawings. Operating interactively within AutoCAD, the detailing system provides the user with total control over drawing production. The details include complete dimensions: piece mark, shape size, cut length, number required, overall length, and clearance distances for each side. Notations can be added.

For more information, contact Norm Ainslie at 518/283-8783 ext. 532 or Circle #99.

SSDCP

A 1995 update of SSDCP's program for producing structural and miscellaneous steel shop drawings is now available. Included are 158 LISP programs running inside AutoCAD R10-13. Programs are available for everything from anchor bolts to roof frames. The program has been used in the field for seven years. A demo disk is available.

For more information, contact Paul Smith at 601/845-2146 or Circle #100.

STEELPLUS

Barry R. Bowen Associates has announced SteelPLUS Structural Shapes R13, an AutoLISP based add-on product for parametrically creating AISC structural shapes in plan, elevation, section and single line. One main dialog controls access to all shapes, allowing multiple insertion points, layer, color, linetype and individual control over the flange and web thickness for details. shape properties are listed in a list box as the shape is selected. The $129 program works with PCs running AutoCAD R12/13 in either DOS or Windows.

For more information, contact Barry R. Bowen at 901/373-6468 or Circle #48.

STAAD3-2D

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

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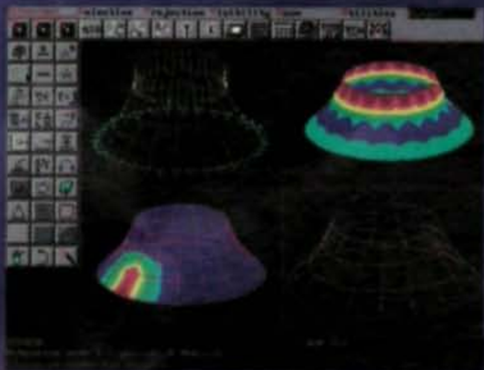

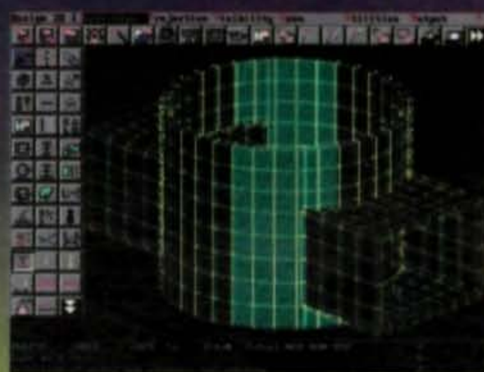

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