MODERN STEEL CONSTRUCTION

November 1992

$3.00

UNITED STEEL DECK, INC.

DECK DESIGN DATA SHEET No. 18

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

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

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

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

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

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

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

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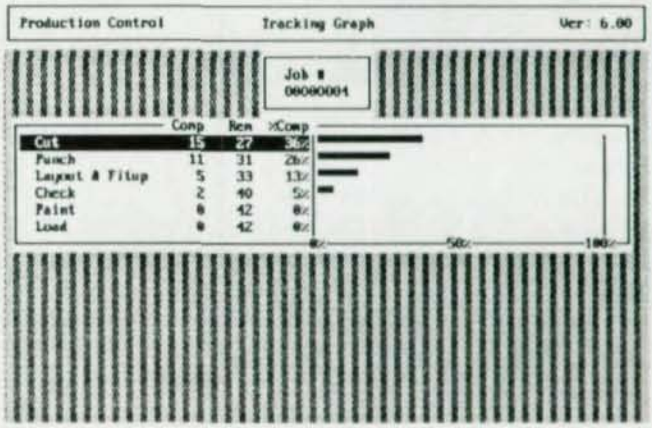

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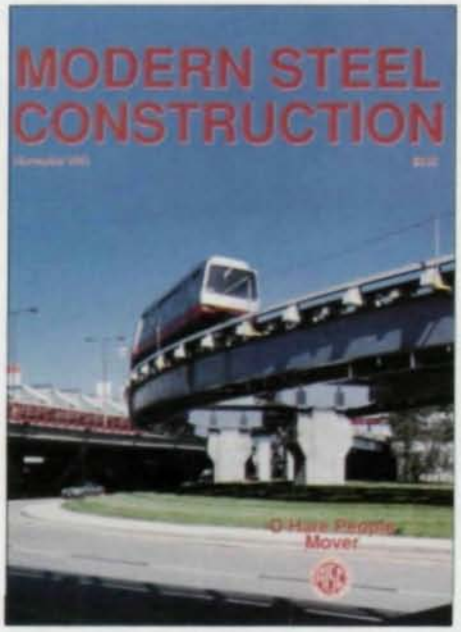
MODERN STEEL CONSTRUCTION

Volume 32, Number 11 November 1992



A new 2.71-mile automated transit system for O'Hare International Airport will transport passengers between terminals and to remote parking areas. Read about this intriguing project beginning on page 16.

FEATURES

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NEWS AND DEPARTMENTS

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Fortunately, Unfortunately*

I was right on deadline. Fortunately, all I needed to complete the issue was to confirm a company's address.

There was a time when this would have been incredibly simple. I would have picked up the phone, called my contact at the company, and asked whoever answered the phone for their correct mailing address. Unfortunately, this call was answered by the dreaded Voice Mail system.

The recording said I could leave a message, or, if I needed to speak to someone, I could press the pound key and dial a specific extension to reach my party's secretary. Fortunately, the information I needed was simple and could be provided by anyone.

I pressed the pound key and dialed the extension. Unfortunately, the secretary also wasn't available.

Fortunately, the Voice Mail system told me I could stay on the line and an operator would take the call shortly.

I waited 30 seconds. I waited 90 seconds. I waited 540 seconds. Unfortunately, an operator never picked up.

I finally hung up and dialed information to get the company's main phone number. Fortunately, the phone company always answers their phone.

I called the main number. Unfortunately, this particular company also uses Voice Mail to answer their main switchboard.

After several more phone calls, I did, fortunately, receive the needed information.

Unfortunately, if I had been a customer calling to place an order or inquire about a product, I probably wouldn't have been so persistent. There are easier ways to do business.

Fortunately, there is a simple solution. Next time you're out with a friend, ask him (or her) to call the office pretending to be a client. Listen in on the call. Find out whether you'd want to be a client or customer of your own company. **SM**

**With apologies to Remy Charlip and the classic children's book,* Fortunately.

Steel Interchange

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

Steel Interchange
Modern Steel Construction
1 East Wacker Dr.
Suite 3100
Chicago, IL 60601

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

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

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

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

An oddity exists when comparing $0.4F_y$ versus $0.3 F_u$ shear stress values. The ratio of F_y to F_u for A36 and A572 Gr. 50 steel is not proportional. For applications based on $F_v = 0.4F_y$, the allowable shear stress for A572 Gr. 50 is 39% greater than A36 steel; however, for applications based on $F_v = 0.3F_u$, the allowable stress is only 12% greater for the A572 Gr. 50 Steel. For A36 steel there is an increase in going from $0.4F_y$ to $0.3F_u$. For A572 Gr. 50 there is a decrease.

a) When a single round hole penetration is required in a beam web, is it proper to use $F_v = 0.4F_y$ or $F_v = 0.3F_u$ when calculating shear capacity?

b) Would a row of bolt holes behave differently than one large round hole which resulted in the same net area?

c) Does the presence or absence of bolts in holes affect the shear capacity of the member?

Mr. Ricker has identified one of the oddities that occurs in the AISC Specifications when more than one limit state (yielding and fracture) must be satisfied. In addition, Ricker also seeks clarification between bolt holes and beams with web openings. Because the inquiry is discussed in allowable stress terms, the response will be based on the June 1, 1989 AISC ASD Specification. The same condition exists in the September 1, 1986 LRFD Specification.

Equation F4-1 ($0.4F_y$) is the limiting stress allowed on the beams gross section to prevent yielding whereas Equation J4-1 ($0.3F_u$) is the limiting stress along the net section that will prevent fracture from occurring through the bolt holes. As Ricker indicates, as the yield stresses increases or the ratio of tensile stress to yield stress decreases (F_u/F_y) the controlling limit state will change from yielding to fracture. Whether this is considered on oddity or a design consideration based on differing limit states is subjective based on the individual engineer's perception. With the preceding information identified the answers to Ricker's three questions are as follow:

a) If one assumes that the hole is of the size intended for a bolted connection (d_b 1½ in.) then both Sections F4 and J4 must be checked and the one giving the lower answer governs. Larger diameter holes suggest a web penetration for an electrical/mechanical duct which should be checked using the provisions of the AISC publication *Steel and Composite Beams with Web Openings*. In my opinion if the diameter of the hole is less than ⅓ of the beam depth, then web opening provisions are unlikely to govern, and the two limit states identified earlier must be checked. As the hole increases in size the web opening provisions and Equation F4-1 must be checked to determine the governing condition.

b) Because bolt spacing is usually three times the bolt diameter (minimum three inches) the maximum material removed by a row of holes is less than ⅓ the beam depth. Under these conditions it is unlikely that there will be any behavioral differences whether there is several holes or one hole whose diameter equals the sum of the individual bolt hole diameters.

c) There have been bolted connection tests indicating that initially there is some variations in connection behavior due to the clamping effect of high strength bolts. However, as the applied load increases and the tightened bolts become loose the difference in behavior is largely undetectable. This suggests that the presence or absence of bolts will not affect the shear capacity of a beam web.

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

The AISC design procedure for end-plate moment connections is for static loading only. Can this connection be used for a utility bridge that has wind loading? Can it be used on a frame that supports a crane runway?

In AISC Design Guide Series No. 4, Extended End-Plate Moment Connections, Chapter 2 - Recommended Design Procedures, only static loading is permitted. However, static loading as it states, includes wind, temperature, and snow. Therefore, the utility bridge subjected to wind loading would qualify as a statically loaded structure and could utilize extended end-plate moment connections. The frame supporting the crane runway may experience many more loading cycles than could be considered as static and should be designed with fatigue in mind.

Craig A. Malony, P.E.
Ballston Spa, NY

Steel Interchange

I have read the July 1992 issue of MSC and find an extremely disturbing anomaly. In response to a query in *Steel Interchange*, "what is a good wind connection for the top of a column," two engineers responded with suggested details. In the same issue, Mr. William McGuire has an article cautioning engineers of the dangers of using prepared material without adequate understanding of the behavior of the structure.

I must agree with Mr. McGuire. The only satisfactory answer to the question asked is: "one that satisfies all of the requirements and costs as little as possible." Another response may lead to inappropriate use of a standard which can easily be done when proper informed thought has not been given to a design.

William J. Gladstone
Consulting Engineer
Manhasset, NY

The AISC design procedure for end-plate moment connections is for static loading only. (See LRFD Manual, 1st Edition, p. 5-143 and ASD Manual, 9th Edition, p. 4-116) Why is this restriction made?

In response to Barry K. Shriver's question regarding the AISC design procedure for end-plate moment connections, I offer the following:

While a designer may be tempted to use the AISC design procedure for end-plate moment connections for any loading combination, a closer examination of the procedure indicates that no provisions for fatigue loading are included. Indeed, repeated loading and unloading even if the yield point is never reached may result in the eventual failure of this connection as a result of fatigue. The procedure apparently considers this possibility by stipulating that the design procedure is only valid for static loading conditions.

The main factors governing fatigue strength are the applied stress, the number of expected loading cycles, the type of detail, and the load path redundancy of the overall structural system. Fatigue need not be considered for a number of cycles less than 20,000 which corresponds to two applications every day for 25 years. Obviously wind loading is not fatigue loading and can be considered a static loading. This leaves high cycle fatigue such as in crane girders, and alternating plasticity as in high seismic regions as areas of concern.

When the beam web and flanges are connected to the end plate by fillet welds, the stresses in the welds are considered to be shear stresses so that the detail is classified as Category F. The allowable stress range in shear is 12 ksi for 500,000 loading cycles (50 applications every day for 25 years). If the beam is connected by full penetration groove welds the detail is Category C for which the allowable stress range in the base material is 21 ksi for 500,00 cycles for the thickness of the flanges and web not greater than 1½ in.

These factors are crucial to the proper design of this type of connection. In the case of Example 39 (9th Edition page 4-120), the designed flange to end-plate connection results in an allowable fatigue stress range of 5.88 ksi for a non-redundant system expecting 500,000 load cycles. Since this example does not differentiate between dead load (static load) and live load (dynamic load), it is impossible to determine if the dynamic stress range will exceed this limit. However as this example clearly illustrates, dynamic loading considerations may well govern the design.

A great deal of judgement is required in determining whether or not dynamic loading will govern the design. It can generally be anticipated that under normal wind loading, fatigue will not be a governing factor. On the other hand, the design of a crane support system may indeed be governed by fatigue. While general assumptions can be made about the type of structure, these assumptions should not replace good sound engineering principles. The AISC design detail outlined in the ASD Manual on pg. 4-116 is useful in performing preliminary designs for structures expecting relatively small stress ranges and a low number of load cycles (less than 20,000). Where this is not the case, an estimation of fatigue strength should be included in the preliminary design. In either case, the consideration of dynamic loading (in conjunction with load path redundancy) should be included in the final design of any structure. Where a high number of cycles and/or a large stress range is expected—as with a vehicular bridge—serious consideration should be given to using bolted connections in lieu of welding.

Daniel G. Faust, P.E.
Ammann & Whitney, Inc.
Philadelphia, PA

New Questions

Listed below are some questions that we would like the readers to answer or discuss. If you have an answer or suggestion please send it to the Steel Interchange Editor. Questions and responses will be printed in future editions of Steel Interchange. Also, if you have a question or problem that readers might help solve, send these to the Steel Interchange Editor.

I am interested in reference sources that address the design of "curved" beams (supporting members rolled to a radius) frequently encountered at the perimeter of buildings, canapies, etc. as well as highway bridges and overpasses. Sources dealing with hot rolled (WF, C, L, etc.) sections are preferred, although any information regarding built-up members would also be appreciated.

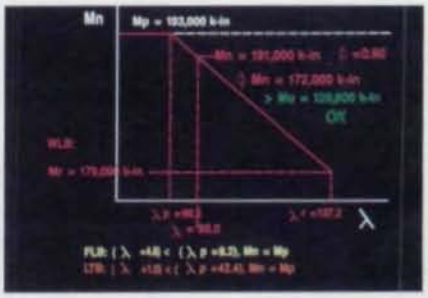
Charles E. Plessner, P.E.
Mound Steel Corp.
Springboro, OH

Correction: The limit in the last paragraph of Don Sherman's response in the September, 1992 Steel Interchange should read $253/\sqrt{F_y}$.

See Ad On Page 42

AISC Introduces Electronic LRFD Specification

AISC is now selling ELRFD, the official electronic version of the LRFD Specification. This sophisticated computer program interactively checks structural steel building components for compliance with the Specification. All provisions of the LRFD Specification are included in the knowledge base of ELRFD, except Chapter I (Composite Members) and Chapter J (Connections).



The ELRFD program checks whether the member satisfies all limit states and limitation requirements set by the LRFD Specification and reports which sections of the Specification are satisfied or violated. Designers can review in detail the formulas and rules in the evaluation and interactively assess any mathematical expression appearing on the screen. Design data produced by the software can be viewed and/or printed in report form for permanent record.

"The program can be used to do sophisticated checking or reviews of designs," explained Abraham J. Rokach, P.E., a senior staff engineer with AISC. "It verifies any steel member from a simple beam to a complex plate girder."

ELRFD was developed jointly by AISC and Visual Edge Software, Ltd., of Quebec, Canada. It is based on research supported by the National Science Foundation. Contributing to the project at various stages have been Profs. Steven J. Fenves and James H. Garrett, Jr. (Carnegie Mellon) and William McGuire (Cornell), as well as members of an ad hoc task group of the AISC Committee on Specifications.

ELRFD has a fully interactive window-based user interface. It runs under Microsoft (MS) Windows 2.0 or higher with MS DOS 3.0 or higher. Hardware requirements are an IBM-compatible personal computer (386 or higher recommended) with a minimum of 2 meg of RAM, a hard disk, a monochrome or color monitor, and a mouse compatible with MS Windows. A printer is recommended but not required.

Included in the ELRFD package are a 3½" diskettes containing ELRFD Checker/Browser, the AISC Structural Shapes Database, and example problems and two manuals (the User's Guide and Reference Manual). Three installations are permitted; the cost of a license is $495.

To order ELRFD or to obtain more information, contact: AISC Software Sales; phone 312/670-5411; fax 312/670-5403. Payment by credit card is accepted.

Bridge Calendar

Bridges 93, a new calendar from the American Society of Civil Engineers, features photographs taken by Eric N. DeLony of the National Park Service. The bridges selected include both historic and state-of-the-art technology. The calendar is available for $8.95 from ASCE Sales & Marketing (SW-16), 345 E. 47th St., New York, NY 10017-2398 (212) 705-7276.

STEEL CALENDAR

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

November 10-11. **CADD Seminars**, Atlanta. Topics include: implementation & management; developing CADD standards; automating the design office; A/E CADD management; linking computers; and training & staffing for CADD. Contact: Sharon Price, A/E/C Systems (800) 451-1196.
Contact: Sharon Price, A/E/C Systems (800) 451-1196.

November 10-11. **Welding Structural *Design two-day seminar,*** *Atlanta*. Designed to provide engineers and welding inspectors a greater understanding of weld mechanics and welded engineering structures. Contact: AWS, 550 N.W. LeJeune Road, P.O. Box 351040, Miami, FL 33135 (800) 443-9353.

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

November 11. **Eccentric-Braced Frame Technology**, breakfast meeting. Overview presentation including how to use EBF in non-seismic areas. Denver. Contact: Jim Anders (214) 369-0664.

November 12-13. **CADD Seminars**, Chicago. (See November 10-11.) Contact: Sharon Price, A/E/C Systems (800) 451-1196.

November 14-19. **SSPC 92**, Kansas City. Nine seminars (40-50 papers), 14 tutorials, and 32 technical committee meetings, plus an extensive product exhibition on steel coatings and paint. Contact: Steel Structures Painting Council, 4400 Fifth Ave., Pittsburgh, PA 15213-2693 (412) 268-2980.

December 3. **Structural Welding Design For Buildings.**, New York City. One day seminar discussing the design of shear, moment, splice and other structural connections, AISC and AWS specification requirements, and other welding issues. Contact: Robert E. Shaw, Jr., Steel Structures Technology Center, (313) 344-2910.

December 6-9. **First World Conference On Constructional Steel Design**, Acapulco, Guerrero, Mexico. (Official conference language is English.) Topics include: Element Behavior & Design; Seismic Design; Composite Structures; Connection Design; and Plate & Box Girders. Contact: Gill Spear, CSD'92 Conference Secretariat Department, Elsevier Science Publishers, Ltd., Mayfield House, 256 Branbury Road, Oxford OX2 7DH, United Kingdom; fax 44 865 310981.

December 7-8. **Central Fabricators Meeting, Union League Club**, Chicago. General topic: engineering and detailing. Contact: LaVerne Duckrow, 7227 W. 127th St., Palos Hills, IL 60463 (708) 361-2338.

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

Jacksonville, FL	November 3
Orlando	November 4
Tampa	November 5
Cincinnati	November 10
Cocoa Beach, FL	November 17
New Orleans	November 17
Boca Raton, FL	November 18
Miami	November 19

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

Wichita, KS	November 10
Denver	November 11
New Orleans	November 17

Steel Buildings: Special Inspection. One-day seminar. Contact: Robert E. Shaw, Jr., Steel Structures Technology Center, (313) 344-2910.

Portland	November 30
Boston	December 1
Cromwell, CT	December 2
Hasbrouck Heights, NJ	December 4

1993 AISC Lecture Series: New Ideas In Steel Construction. Seminar covers four topics: low-rise construction; *Manual Of Steel Construction, Volume II* (Connections); eccentrically-braced frames; and Partially-Restrained Connections. Contact: Robert Lorenz, AISC, One East Wacker Dr., Suite 3100, Chicago, IL 60601-2001 (312) 670-5406.

West

San Diego	February 18
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Los Angeles	May 6
Irvine	June 10
Sacramento	June 22
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Kansas City	April 15
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Knoxville	September 16
Atlanta	September 28
Wilmington	September 30
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Richmond	October 28
Orlando	November 9
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Northeast

Meriden	January 26
Boston	January 27
Portland	January 28
Newark	April 6
New York	April 28-29
Rochester	September 22
Albany	September 23

Mid-Atlantic

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Cleveland	May 13
Columbus	October 20
Cincinnati	October 21

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	A36	CHAPARRAL A36/A57250	A572-50
Yield Point, ksi	36 (min)	50 (min)	50 (min)
Tensile, ksi	58 - 80	65 - 80	65 (min)
Elongation in 8"%	20 (min)	20 (min)	18 (min)
Elongation in 2"%	21 (min)	23 (min)	21 (min)
Carbon %	0.26 (max)	0.22 (max)	0.23 (max)
Manganese %	——	0.50 - 1.35	1.35 (max)
Phosphorus %	0.04 (max)	0.04 (max)	0.04 (max)
Sulphur %	0.05 (max)	0.05 (max)	0.05 (max)
Silicon %	0.40 (max)	0.40 (max)	0.40 (max)
Columbium/Niobium or Vanadium %	——	0.005 - 0.05*	0.005 - 0.05

* If material is processed with vanadium, the typical vanadium range is .010 – .030.

	44W	CHAPARRAL 44W/50W	50W
Yield Point, ksi	44 (min)	50 (min)	50 (min)
Tensile, ksi	65 - 90	65 - 80	65 - 90
Elongation in 8"%	20 (min)	20 (min)	19 (min)
Elongation in 2"%	23 (min)	23 (min)	22 (min)
Carbon %	0.22 (max)	0.22 (max)	0.23 (max)
Manganese %	0.50 - 1.50	0.50 - 1.35	0.50 - 1.50
Phosphorus %	0.04 (max)	0.04 (max)	0.04 (max)
Sulphur %	0.05 (max)	0.05 (max)	0.05 (max)
Silicon %	0.40 (max)	0.40 (max)	0.40 (max)
Columbium/Niobium or Vanadium %	0.10 (max)	0.005 - 0.05*	0.10 (max

* If material is processed with vanadium, the typical vanadium range is .010 – .030.

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O'Hare's new transit system uses automated trains to rapidly transport passengers between terminals and remote parking areas. Photo by Abby Sadin.

People Mover Flies On Steel Wings

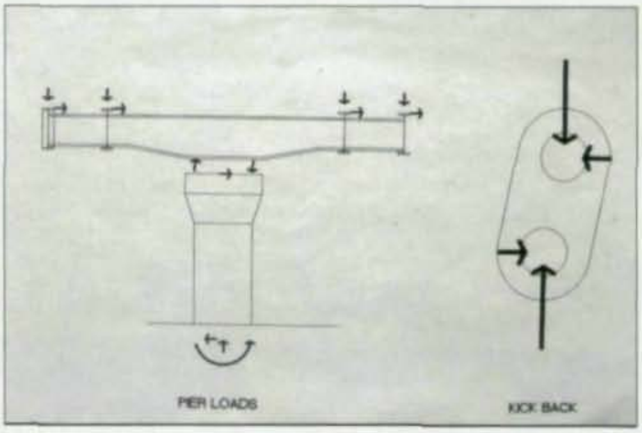
Wing-shaped crossheads minimize the depth of the supporting structure for a new transportation system at Chicago's O'Hare International Airport

By James M. Daum, P.E., and Steve G. Citko, P.E.

Along with its reputation as the world's busiest airport, Chicago's O'Hare International Airport is well known for its congestion. But by the end of 1992, movement between the airport's terminals and remote parking facilities will be both greatly simplified and much accelerated thanks to a new automated transportation system.

The 2.71 mile, $150 million MATRA VAL Automated Guideway Transit (AGT) system features driverless trains capable of traveling at 50 mph. The system will conveniently connect domestic and international terminals to each other and to remote parking facilities. To alleviate congestion, the People Mover is designed to carry up to 2,400 passengers an hour with only 90 second intervals between trains.

The guideway system, designed by Chicago-based Teng & Associates, includes more than 10,000′ of elevated structure, 4,000′ of slab on grade construction, and 4,800′ of track in a storage and maintenance facility. Three pedestrian bridges conveniently link the People Mover stations, the terminals and the parking garage. The pedestrian bridges are designed as a single, 160′ span through truss. The through truss is primarily a Pratt truss, but it combines a Vierendeel panel at the area where the stations connect to provide pedestrian access.

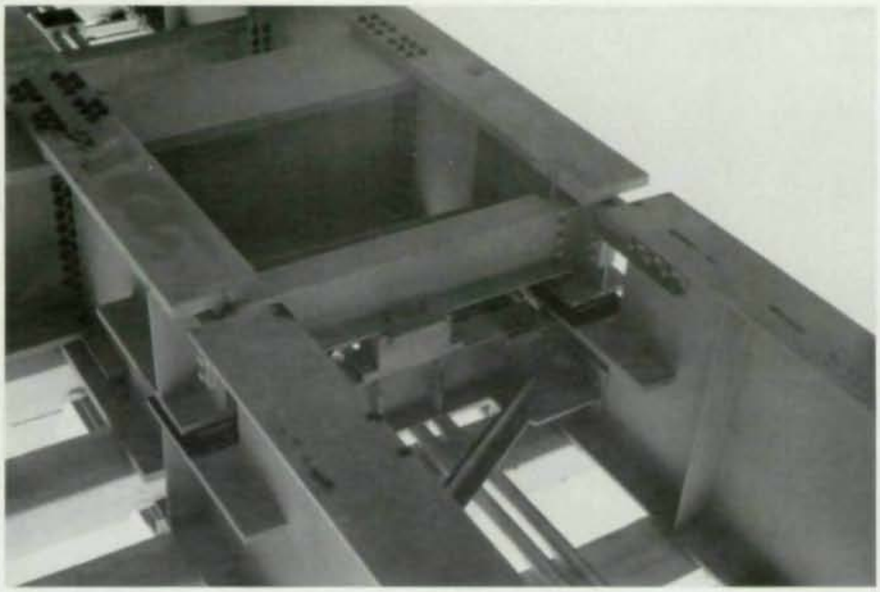


Above- and below-ground congestion necessitated wide-spaced guideways with narrow piers. The guideways are supported by wing-shaped crossheads that pierce the web of the inner guideway girders.

Space Constraints

For most of the AGT system, two guideways (inbound and outbound) were constructed with a close-spaced alignment with 13′ between centerlines. However, severe space constraints due to existing structures in the core area near the domestic terminals required an innovative engineering solution for the guideway and its supporting system.

Horizontal placement of the guideway had to fit in a narrow band of space available between the existing bi-level arrival/departure roadways and the six-level parking garage. A center station boarding platform with the



A shelf joint provides thermal expansion and transfers lateral loads down from the bracing level through a shear transfer device and back up to the bracing level.

guideway wide-spaced at 31′ apart between centerlines was the narrowest functional arrangement that permitted sufficient boarding area and still fit within the available space. Because of the location of the lower level roadway, a single column system was required to support the guideways, platforms and station. A framed bent support system would have placed the columns in the roadway.

In addition, vertical placement of the guideway had to allow vehicle access on the roadway below and provide clearance for People Mover vehicles underneath overhead pedestrian bridges connecting the terminals to the stations. To accomplish these requirements, the guideway girders had to frame into the supporting crosshead.

The end result are wing-shaped crossheads designed to allow the structure to overhang or "fly over" the lower roadway.

Steel Superstructure

Both steel and concrete were considered during the design phase. However, because of its lighter overall system weight and its versatility to handle 200′ radius curves, steel was selected.

The design features an open deck track structure using prestress

A map of O'Hare International Airport shows the 2.71-mile path of the Automated Guideway Transit system connecting terminals, rental car return, and remote parking.

concrete ties on 5′ centers, which allows snow to pass through its structure and helped to reduce weight. The track system was supported by guideway spans comprised of two 50"-deep A36 plate girders with 14"-wide flanges varying from ¾" to 2½" thick. Since the rubber tired vehicles had relatively low impact forces, the flanges were attached to the webs with fillet welds rather than full penetration welds.

The ties were located where the lateral bracing system connected with the main supporting girders. This coordination helped transfer the lateral loads to the lateral bracing rather than through the ties to the girders, which would have exerted excessive loading to the weaker axis of the girders. The lateral bracing system was comprised of WT section members located at the top of the diaphragm level.

Torsional effects were resolved by W24 diaphragms connected to interior stiffeners spaced 15′ on tangent and 10′ on curves. The diaphragms were set several inches below the top of the flange of the guideway to create a space for electrical conduits to be carried on top of the diaphragms.

Superelevation, or banking, was achieved by adjusting the elevation

The People Mover project included the construction of stations in several locations.

of the guideway girders and using tapered shim plates to accommodate for rotation of the tie. Superelevation, vertical curves and horizontal curves were achieved by accurately measuring and cutting the web and flanges prior to welding.

Analysis Studies

The beams were sized using a simple analysis that treated each guideway as an independent structure. Train locations were determined to produce maximum moment, shear and deflection. Additional forces due to curvature effects were accounted for by a V-Load type of analysis. Adequate section size was then checked using an interaction equation.

Analysis was performed using STAAD-III from Research Engineers, Inc.

Forces from design were compared with results from two detailed frame analysis studies. The sharpest curved segment in the wide-spaced and close-spaced areas were modeled using up to 700 members. These models were used to study in more detail the curvature effects and interaction between the guideways as a system. Although the system exhibits these more complicated types of response, the magnitude of forces were comparable or less than those obtained from the more simplified design procedure.

Forces and displacements from these studies were stored in a large database. With 700 members, 350 nodes, 20 load cases and several train positions, the number of forces and displacements was in the hundreds of thousands. When designers wanted to look at values for one type of member or response at one location, however, they were able to extract by computer a small subset of information. They could then look at only a few pages of output rather than looking through hundreds of pages of standard output.

Crossheads

The "flying" crossheads were required to meet several objectives, including: resisting the overturning forces from wide-spaced guideways and vehicles; supporting the passenger stations; and transferring longitudinal guideway forces to the piers.

The height restrictions discussed earlier necessitated that the guideway girders be framed into the supporting crossheads. The wing-shape reduced the crosshead depth at the ends to let the crosshead pass through a reinforced opening in the web of the inner guideway girders while keeping them deep in the center for strength. This allowed guideway girder splices to occur near quarter points and not at the crossheads, keeping the crosshead connections very clean.

The transition in depth of the crosshead was achieved by two reverse curves in the bottom flange. This provided an aesthetic design and also eliminated butt welding of flanges up to 3½" thick since the gentle curves instead allowed the use of a single continuous flange plate.

The bottom flange was thicker so that it could resist longitudinal forces from the guideway in bending and transfer the force to the piers. Crossheads were fabricated from A588 steel, and the flanges were attached to the webs with full penetration welds while stiffeners were fillet welded.

Special Considerations

Due to the lack of redundancy in the structure, most components are fracture critical, and thus fatigue became a major influence in the design. The design of components was based on stress ranges appropriate for the number of cycles anticipated. Fatigue sensitive details were avoided. Large radii were specified at the re-entrant corners of the shelf joint and at the pierced guideway girder web to avoid stress concentrations.

Field welding was almost totally eliminated in favor of bolted splices and connections. A fracture critical control plan for welding was carefully followed that influenced types of welds and the welding procedures permitted.

All of the track and structural components have details to allow thermal movement at expansion joints, which are spaced 300' apart or less. The rails have finger joints. The steel girders have a shelf arrangement that provides support for the span in addition to allowing for thermal expansion.

Special link bearings supporting the crosshead were developed to accommodate uplift forces of 300 kips, while allowing for the necessary rotation and expansion that the structure and crossheads undergo when loaded. Because portions of the bearing are up to 4" thick, A486 Grade 120 cast steel was used to alleviate welding and fatigue problems. While the bearings are designed for vertical and longitudinal forces, lateral forces are taken by a central shear transfer device.

Piers And Caissons

Congestion was not just a problem above ground. Below ground in the core area are utility lines, walkways, offices and parking. The piers were placed wherever practical to work around all of the obstructions. As a result, some piers are as close as 50′ while others are more than 100′ apart. Because of the space restrictions due to roadway clearance, the single column piers are only 6′ wide. Therefore, they needed to be constructed of 9,000 psi concrete and were post-tensioned to reduce deflection and resist overturning moments of up to 3,200 k-ft.

Post-tensioning was achieved using either bars or cables depending on the magnitude of force, which could be as high as 4,000 kips effective prestress. Because of the large uplift on the bearings, they were held down by four post-tensioning cables per bearing. Main post-tensioning bars and cables were grouted, while bearing cables were left ungrouted and curved to allow side access for future removal, if necessary.

The soils consist of a layer of loose granular material followed by stiff-to-hard clays, with some dense sand and silt. Most piers are supported on a single 8′- to 9′-diameter caisson, which was designed to meet strength and deflection criteria. Caisson lengths range from approximately 30′ to 55′. Where utilities and structures were immediately adjacent, making the top portion of the caisson ineffective, a tripod or T-shaped arrangement using three smaller caissons or a grade beam with two caissons was employed instead.

Team Members

The People Mover was designed and built by the French company MATRA. ICF Kaiser Engineers was the supervising consultant for architectural and engineering services. The guideway structure and pedestrian bridges were designed by Chicago-based Teng & Associates Inc., a full-service E/A firm. Structural steel fabricator was AISC-member Pitt-Des Moines, Inc. Erection was by Hi-Gate Erectors, Inc.

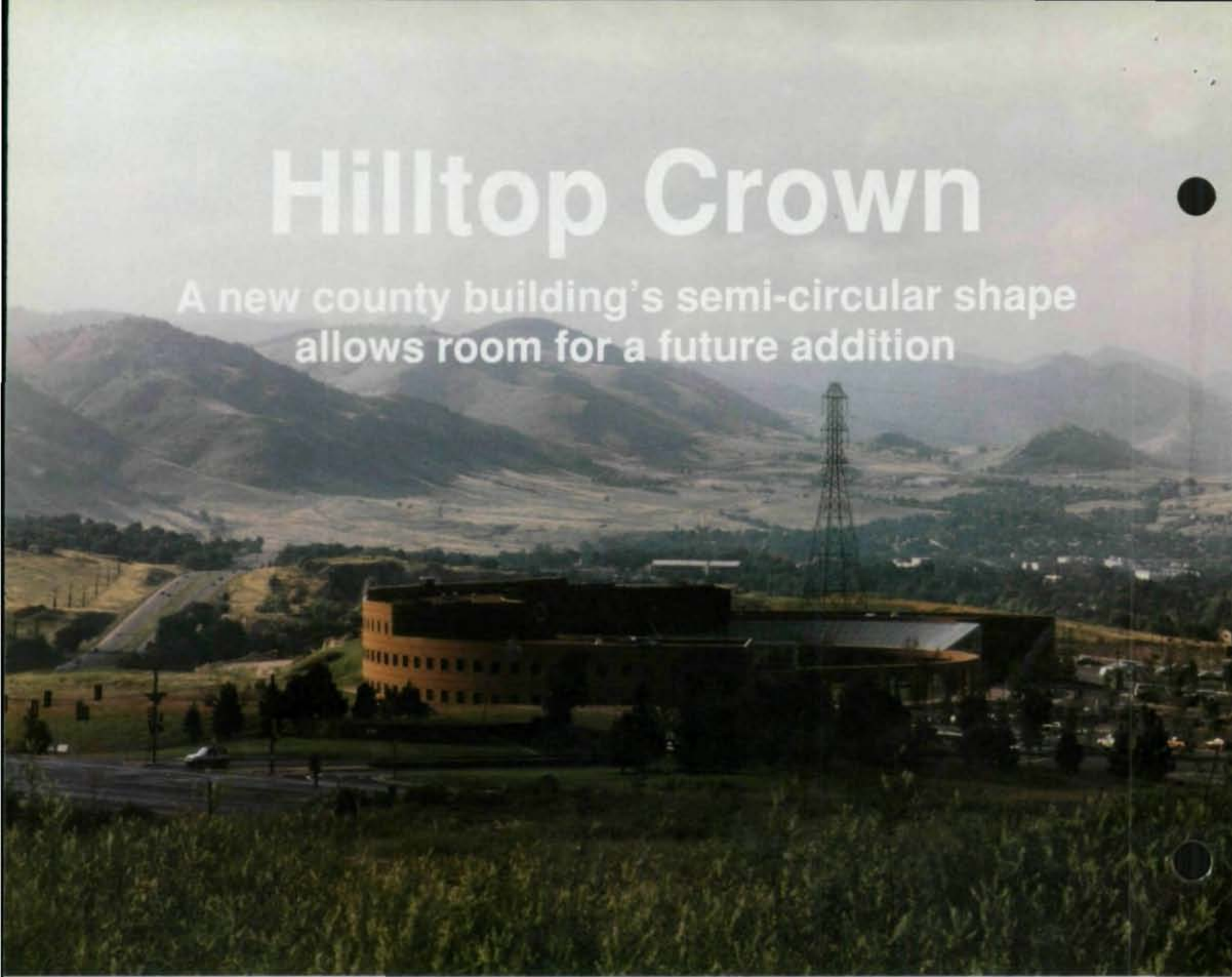
James M. Daum, P.E., is a vice president and Steve G. Citko, P.E., S.E., is a structural engineer with Teng & Associates.

WHEN JOISTS SAVE
THEY CALLED IT TH

The "Duke" installing ceiling panels in the Hanging Lakes Tunnel, Glenwood Canyon, Colorado.

Hilltop Crown

A new county building's semi-circular shape allows room for a future addition



By Richard Weingardt, PE

Steel's versatility was put to the test in the design of the $9.5 million Jefferson County Human Services Center. The 134,000-sq.-ft. office building in a fast-growing county between Denver and the mountains just to the west, is a three-quarter circle in design. Additionally, the structure had to accommodate soil movements, wide temperature fluctuations, and heavy winds.

Located on a hilltop slightly more than 6,000' above sea level, the circular structure surrounds a large courtyard open to the east. The site, planned as part of a 196-acre governmental complex for the county, already contained several buildings and shortly all county government services will be relocated to the site. It's location in the heart of the county, combined with the panoramic view of the city to the east and mountains to the west, make it a natural choice for such a prominent complex.

The three- and four-story-high structure is a three-quarter circle in plan and fans out around a central courtyard. It is 360' in diameter and forms a large three-dimensional horseshoe. The design accounts for the possibility of a future addition, which is planned as an 80,000-sq.-ft. segment completely closing the circle.

Though the site is in a seismic Zone I area, earthquakes are not a major structural design factor in Colorado. Expansive soils and climatological conditions are, however. They dictated some unusual measures be taken in the design of the building to provide resistance to extremely high and violent winds, heavy snows, soil movements, and temperatures varying more than 125° F.

Structural System

The structural skeleton of the building is comprised of A572 Grade 50 rolled high-strength steel sections. Lateral resistance is provided by a combination of X-braced frames and core walls. The braced frames consist of A36 steel tube members. Composite steel and concrete floors—lightweight concrete topping and steel decking over steel beam and girders—provide the necessary diaphragm action against lateral forces. The roof deck system is similar, except there is no concrete topping and open web joists are used instead of beams.

Typical floor beams are W18x35 at 10' on center for the outside bays and W16x31 at 8' on center for interior bays. Perimeter girders are W21x44 and W24x55. Roof joists are 18H7 at 4' on center for both interior and exterior bays. Typical interior columns are W10x40 while those in the exterior walls are W10x33.

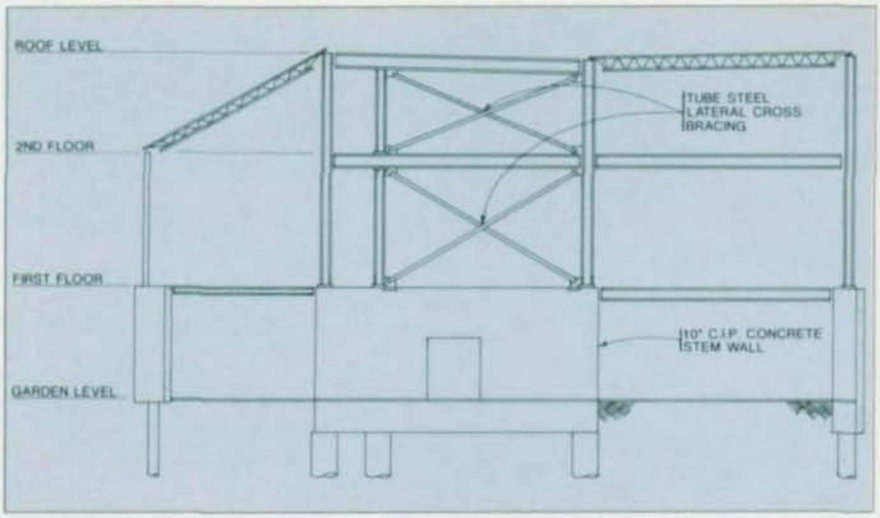
Due to the semicircular plan of the building, each steel component had to be individually fabricated and fit into place along the circular grid lines which made up the flowing architectural envelope. Contractor on the project was Pinkard Construction Co., Lakewood, CO.

Unique Criteria

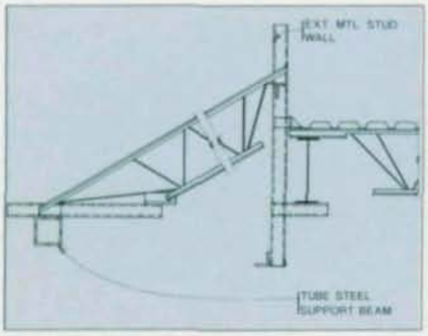
Structurally, the circular building presented several unique design problems. Since thermal and soils movements—both lateral and vertical—were major concerns, the structure was designed in three segments with two expansion joints between. Double columns were not used at these joints; rather, a single column with slip joint haunches was designed. Teflon pads were used between beams and haunches for smooth action during movements.

Construction itself phased around the design and expansion joints. The building was erected in three vertical sections rather than by the traditional floor-by-floor method. This allowed work to pro-

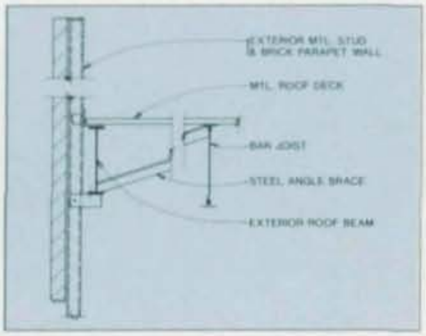
The 132,000-sq.-ft. circular building wraps around a colonnaded plaza, creating a landscaped courtyard as a transitional space between the building, its parking area and the natural terrain. As befits a building in nature-conscious Colorado, the designers incorporated the colors and textures of the landscape. Wheat colored brick is used for the exterior walls, to match the color of the clay in the foothill hogbacks, and bands of deep maroon granite outlining the brick draw from the colors of the surrounding terrain. Photos opposite and at top by Nick Merrick, Hedrich Blessing. Construction photos courtesy of Pinkard Construction Co.



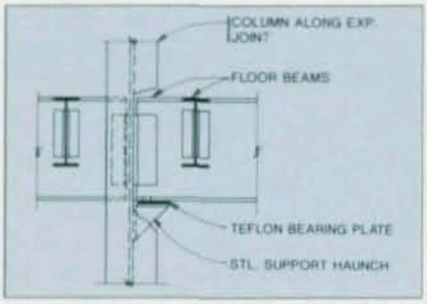
Building section showing combination steel X-bracing and concrete shear walls for lateral resistance.



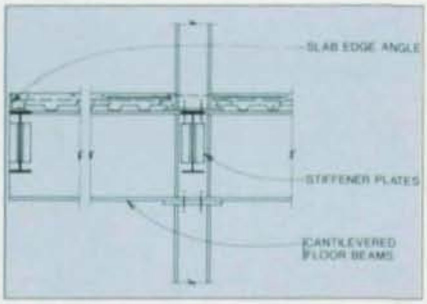
Exposed steel framing to support roof skylights circling around the courtyard walls.



Steel framing detail for parapet wall at the roof.



Expansion joint using single column with bracket and teflon slippage detail.



Cantilevered floor beams at circular exterior walls.

ceed on respective sections as the materials became available and each vertical section could be constructed at its own pace.

The project's steel fabricator and erector were put to the test throughout construction. "The fabricator had a great deal of challenge relative to the layout of the building in general," explained John Davis, vice president in charge of the project for structural engineer Richard Weingardt Consultants.

"Many of the features of the building were not laid out in a radial fashion, yet the grid lines were—two sets of ground rules for the same structure. Construction was done in three independent curved segments, with each being laterally independent as if it were a separate structure, yet that lateral design had to be compatible with the adjacent segment, and tied together when completed so you wouldn't get three buildings moving independently—and differently—when exposed to lateral forces."

The building foundations consist of concrete piers drilled into bedrock, a sometimes difficult job because the irregular "Red Rocks formation" of the foothills area has many seams and cracks.

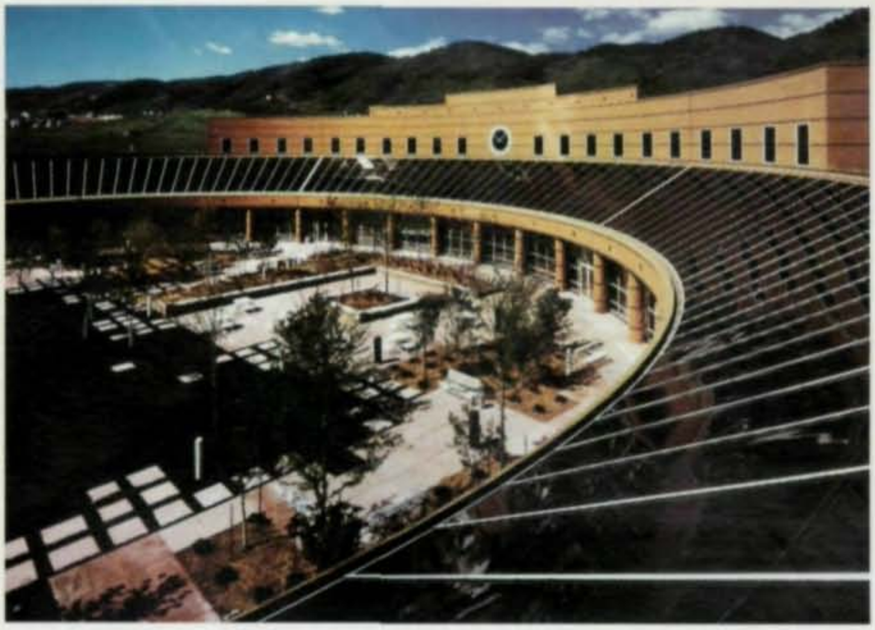
Design Parameters

The architectural design included the need for high screen walls to conceal the roof top units from view. The structural capacity of these parapet walls, as well as the steel structure itself, was well tested during construction when on several days the temperature fell to 25-below-zero and the wind reached 90 mph.

The curved exterior walls are clad with brick made from carefully selected clays that blend in with the surrounding environment. Additionally, the architect, C.W. Fentress J.H. Bradburn Associates, Denver, used Italian granite articulation bands throughout to create relief for the large expanses of brick.

Interior partitions radiate outward from the curved courtyard walls like the spokes of a wheel.

A two-story solarium rings the inside wall of the building, providing circulation and views of the landscaped courtyard below. Reflective glass on the solarium's skylight cuts glare and heat. The terrazzo floor is patterned in two colors, emphasizing the colonnade and the radial pattern of the space itself. Photo at right by Tom Travis. Photo at right by Nick Merrick, Hedrich Blessing.



The unique room configurations created challenging ceiling and floor covering layouts, especially in the computer room where raised access flooring was utilized.

Around the plaza courtyard area, skylights rest on a complicated exposed steel joist roof framework. Daily elongation and contraction cycles caused by daytime heating and evening cooling of the brick and steel framing members are controlled by a series of structural expansion joints that separate the building into manageable segments.

"The building [which has been in use for nearly a year] has done extremely well," said Edward Balkin, project designer for Fentress/Bradburn. "The steel system has more flexibility for us to do the curvilinear shape."

Construction of a massive new \$58 million, 521,000-sq.-ft. Judicial and Administrative Center, designed by the same architects and engineers, is now underway.

Richard Weingardt is president of Richard Weingardt Consultants, Inc., a large Denver-based structural engineering firm. ■

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Baseball's Past Revisited

While nostalgia played a part in choosing steel for Baltimore's new stadium, economics also was an important consideration

Photography by Janice E. Rettaliata, Baltimore

By John S. Palmer

When Oriole Park at Camden Yards opened in March, critics raved. "[Oriole Park] is a building capable of wiping out, in a single gesture, 50 years of wretched stadium design and restoring the joyous possibility that a ballpark might actually enhance the experience of watching the game of baseball," wrote architecture critic Paul Goldberger in the New York Times.

And after a full season of play, the reaction is still just as positive.

Much has been written and said about this 47,000 seat facility built on an 85-acre site in an historic neighborhood. It is a ballpark that takes its character from the surrounding streets and the building materials chosen to construct it: the open airiness of exposed structural steel trusses and a facade of red brick and architectural precast concrete panels.

According to Bruce Hoffman, executive director of the Maryland Stadium Authority, the Orioles wanted to build in steel to shape a facility reminiscent of some of the colorful, human-scaled ballparks of the past, such as Fenway Park in Boston, Wrigley Field in Chicago, and Brooklyn's old Ebbets Field. What they envisioned was a "field of dreams" where heroes were made and crowds became devoted fans—an antithesis to the massive concrete cookie-cutter all-purpose facilities that dominated the sports construction industry in the 1960s and 1970s.

"From the start, the Orioles wanted to use steel because it would be more appealing and friendlier," said Janet Marie Smith, the ballclub's vice president for stadium planning and development. "We also wanted the stadium to fit the context of historic Baltimore, which is an old steel town."

Lehrer McGovern Bovis (LMB) was retained in January 1989 to act as project advisor for the Baltimore Orioles and to oversee the technical aspects of the project, and later as construction manager for the Concessionaire, ARA Services. Barton-Malow/Sverdrup, Upper Marlboro, MD, was the project's construction manager.

"The design was an evolving process that involved many in the key decisions," said Ben Barnert, senior vice president with the architect, Hellmuth, Obata & Kassabaum (HOK Sport), Kansas City.

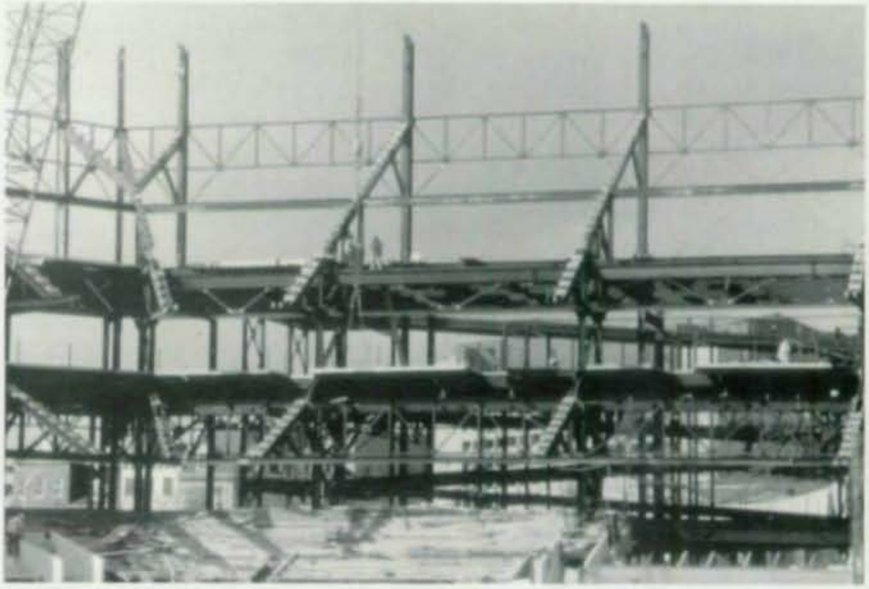
Economic Design Considerations

Early on, it looked as though concrete would be the more economical building material and the design proceeded based on that belief. However, because the Orioles wanted a more traditional and more friendly design, the design team took another look at steel.

Ken Hiller, vice president and chief structural engineer at LMB, recalls: "The data based on the original steel design versus concrete indicated that it would cost 50% more in steel. Based on my experience, I knew it was impossible for there to be that great a differential."

Several consultants were brought in to rework the calculations, including Paul Rongved, Ph.D., of Rongved Associates, New York City, an expert in the field of computer modeling. He contended that the original data input was incorrect and that a steel truss system would actually be less expensive than the concrete alternative.

"One of the problems in the calculations was they overlooked the expansions," explained Rongved. "The truss was calculated as a continuous member without taking into account that it was hinged at each truss point."



Once it was determined that steel was economically feasible, the question became whether the stadium could be redesigned with steel and still built in time to open the 1992 season.

LMB carefully analyzed the erection schedule using a three-color computerized graphic scheme to depict crane sequencing and the daily schedule of crane coordination and determined that two months could be gained within the overall construction schedule.

Bliss & Nyitray, Miami, who was later brought in as the project's structural engineers, concurred with Rongved and LMB that steel would be as economical as concrete but more aesthetically pleasing—as long as maintenance concerns could be satisfied.

The final design features a truss configuration that appears at the back row of the lower seating bowl and rises up through the entire facility. The main support bents, spaced 42½' on center, provide both lateral bracing for the stadium and support for the private box level bleachers and the upper level bleachers by means of 36' and 21' cantilevers, respectively.

The concourses and the enclosed club are supported by W24x68 floor beams spaced on 9' centers framing to the bents, and are composite with the 2½" slab poured above the 3"-deep, 20 gauge composite steel floor decking. All structural steel is ASTM A36 and all bolted connections used ⅞"- and 1⅛"-diameter A325 high strength bolts.

Maintenance Concerns

To reduce corrosion and maintenance costs of the exposed steel in this coastal salt-air environment, the steel was shop painted with an organic zinc rich primer and a high-build aliphatic polyurethane finish coat. The perimeter of all exposed welded connections were seam welded to prevent moisture intrusion. The faying surfaces of bolted connections were primed with an inorganic zinc rich primer that provided the necessary slip coefficient for the slip critical connections.

Waterproofing of the exposed concourse slabs was achieved by a waterproof membrane installed on top of the structural slab and then topped with a nominal2"-thick slab reinforced with polypropylene fibers.

The private box level and upper level seating are framed with precast prestressed double "L" bleacher units. The structural behavior of the "folded plate" units was measure by a full scale load test that verified the design exceeded the requirements for load capacity, deflection and crack width.

John S. Palmer is director of sports facility practice with the Washington, DC, office of Lehrer McGovern Bovis, Inc.

Structural Considerations At Camden Yards

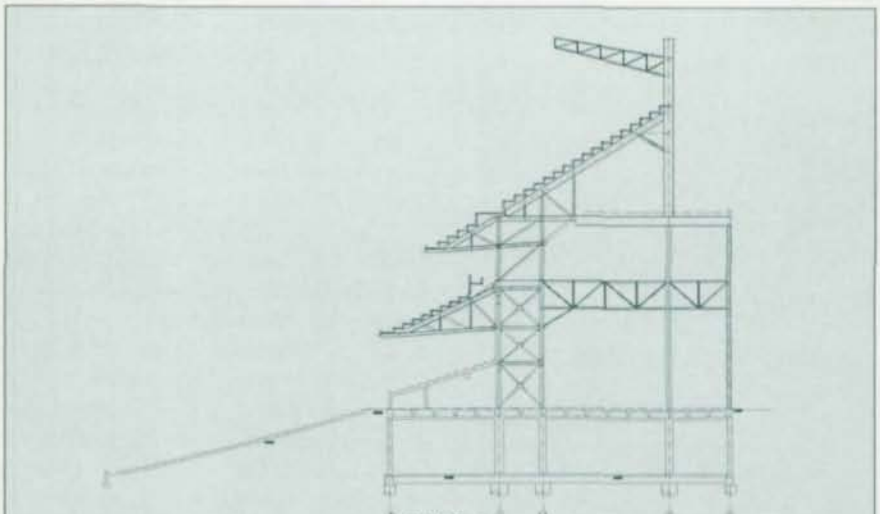
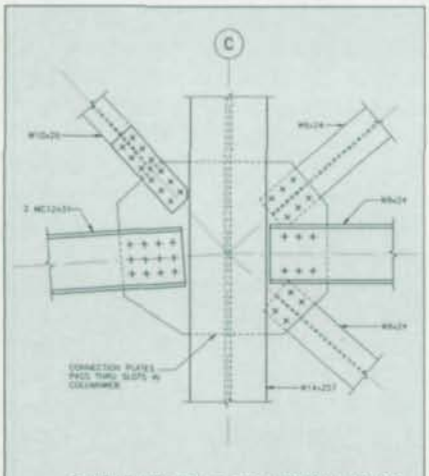
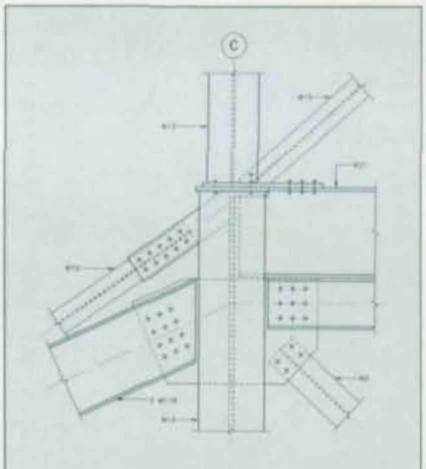


FIGURE 1



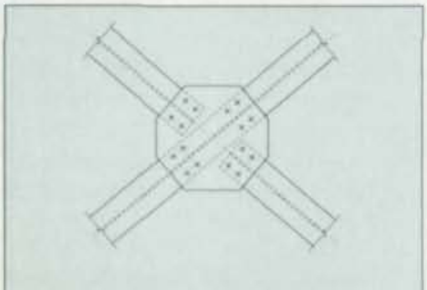
CANTILEVER TRUSS TO VERTICAL TRUSS BOTTOM CHORD CONNECTION DETAIL

FIGURE 2



CANTILEVER TRUSS TO VERTICAL TRUSS TOP CHORD CONNECTION DETAIL

FIGURE 3



TYPICAL DIAGONAL CONNECTION DETAIL

FIGURE 4

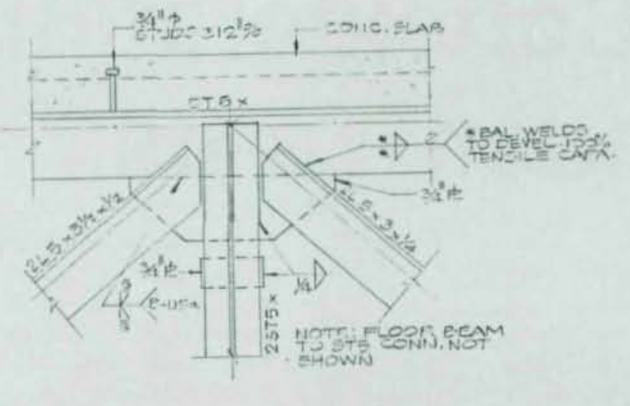
By Wm. Barton Wallis, P.E.

The Stadium above the Main Concourse Level is framed in structural steel. A design and cost comparison analysis was conducted between using A36 and A572 Grade 50 steel and at the time of the comparison, the A36 steel proved to be slightly more economical when comparing minimum depth and weights, deflections during concrete placement, and floor vibrations.

The main support bents, spaced 42'-6" on-center are primarily a truss configuration (see Figure 1) and provide both lateral bracing and gravity support for the Stadium. The primary bent employs a 12'-6"-deep vertical truss. The vertical truss chords are W14 sections with typical weights of 257 lbs. and 120 lbs. located at Grid Lines "C" and "C.3" respectively. The diagonal web members are W8x24 and the horizontal web members are double channels, which facilitated the connections.

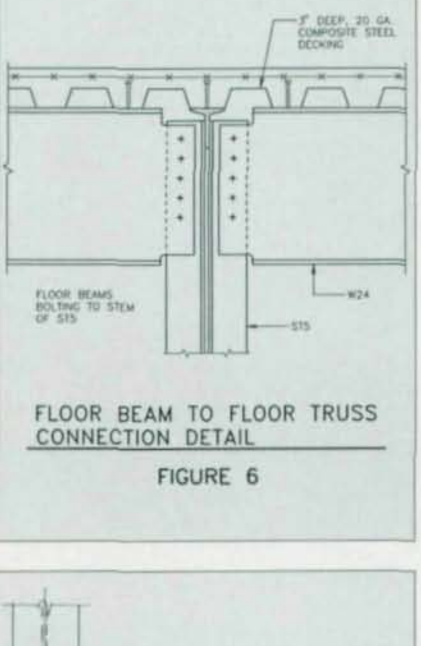
Double plate connections were used in which the plates were placed on the outside of and bolted to the flanges of the W8 diagonals. The webs of the horizontal channels were then bolted on the outside of the plates and bolted as shown in Figures 2 and 3. The plates were shop welded to the webs of the W14 chords.

The Private Box Level bleachers and the Upper Level bleachers are supported, respectively, by means of 36' and 21' cantilever trusses. The web members are W10x26 members placed between the double channels with the flanges of the W10s bolted to the webs of the channels (see Figure 5). This proved to be a very simple truss to fabricate and provided a very stable and efficient member. Steel fabricator was AISC-member Trinity Industries Inc.



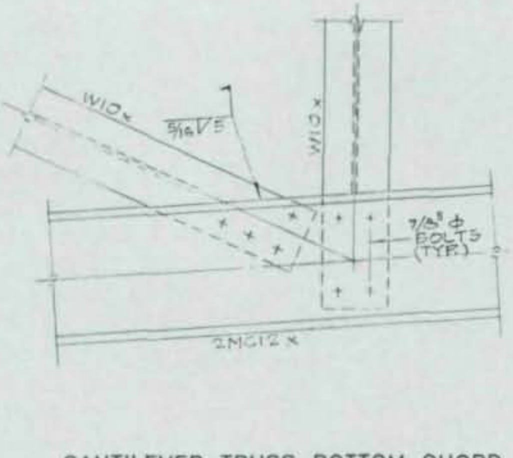
FLOOR TRUSS TOP CHORD PANEL POINT DETAIL

FIGURE 7



FLOOR BEAM TO FLOOR TRUSS CONNECTION DETAIL

FIGURE 6



CANTILEVER TRUSS BOTTOM CHORD DETAIL

FIGURE 5

The vertical truss and the cantilever trusses were each prefabricated and bolted together at the site. The connecting plates transferring the extremely large compression and tension forces extended through slots in the webs of the W14 columns, greatly reducing sensitive welding. The top and bottom chord connections are shown in Figures 2 and 3.

The private box level is framed with an 8'-deep truss connected to the vertical truss at Grid Line "C.3" and to W14 columns at Grid Lines "D" and "E". ST6x25 sections were used for the top and bottom chords, double angles for the diagonal web members and double ST5x12.7 vertical web members. The floor beams are spaced on 9'-½" centers and are connected directly to the ST5 vertical members with a standard shear connection as shown in Figure 6. This system eliminated the need for added plate connectors, simplifying fabrication. The chord stems were extended with a plate where the highly stressed diagonal members connected as shown in Figure 7.

The floor beams at the Upper Concourse level also are spaced on 9'-½" centers and are connected to W33x141 girders with single plate shear connectors.

The floor beams at the Private Box Level and the Upper Concourse Level were typically W24x68 members designed and detailed to be partially composite steel floor decking with 2½"-thick, 4,000 psi regular weight concrete topping providing a total depth of 5½". The structural slab was then topped with rigid insulation and a 2" non-structural topping at the Private Box Level and by a waterproof membrane and 2½" to 4½" non-structural topping sloped to floor drains at the Upper Concourse. The topping slabs were reinforced with polypropylene fibers.

At Homeplate, the Upper and Lower Press Levels were added between the Main Concourse Level and the Private Box Level. The additional two levels restricted headroom and thus the trusses were replaced with rigid frames.

The double "L" precast prestressed Upper Seating bleacher units are supported on steel brackets bolted to W24x76 raker beams. The raker beams are connected at the top end to W33 columns extending up to support the cantilevered sun canopy and, above that, the field lighting support trusses.

The sun canopy is framed with planar trusses interconnected with diagonal bracing giving the appearance of a space frame. The trusses are fabricated with WT5 and WT7 chord members, single angle diagonal members and single channel vertical web members. The chords of the typical canopy roof trusses are then simply bolted to the web of the vertical channels of the cantilever trusses and the diagonal bracing, consisting of single angles, is bolted to the web of the canopy roof truss members.

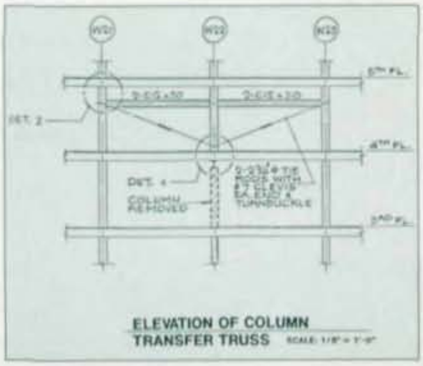
Transfer trusses are located throughout the Stadium at the Field Lighting, below the top row of the Upper Seating, and below the Private Box Level Seating. These trusses, most of which are visible from the exterior of the Stadium, greatly contribute to the aesthetic appearance. Also, by connecting the top and bottom chords to the columns, they create rigid frames, which provide lateral stability in the transverse direction.

The proper selection of the members was crucial in achieving economies in fabrication on this project. The members were selected not only for their structural performance but also for their connectability and aesthetic appearance.

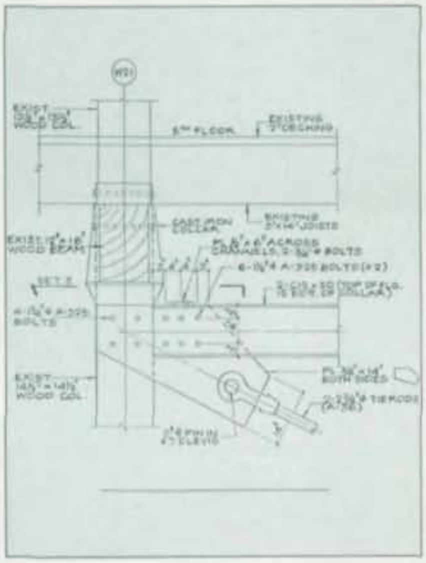
Warehouse Renovation

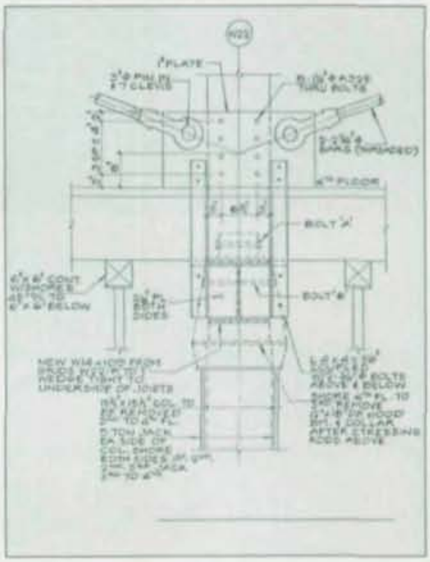
While most attention has been focused on the stadium, restoring and renovating the eight-story warehouse beyond the right field wall was also a challenging structural project.

The 50'-wide, 1,100'-long building was originally constructed of heavy timber with 3'-thick bearing walls founded on cut stone wall footings bearing approximately 13' below grade level. The floors are composed of two layers of 1"-thick oak decking supported by 3"x14" wood joists on 18" centers spanning 17' to heavy timber girders. The girders span 17' to cast iron columns at the lower floors and heavy timber columns at the upper floors. The column's foundations are tiered cut stone blocks.



Structural considerations in the renovation of the old warehouse structure.





The building houses the team's administrative offices, restaurants, a stadium club and other facilities, all of which are connected to the stadium structure by a tunnel to the service level and an open pedestrian bridge to the private box level.

To provide a column-free space for the Oriole's new boardroom, a column was removed between the third and fourth floor after the 95-ton column load was transferred to a new support truss. The new truss was installed between the fourth and fifth floors and is framed with a pair of C12x30 channels bolted to gusset plates and the existing heavy timber columns, directly below the existing fifth floor framing members.

Creating the column-free space was quite complicated. A structural steel collar was fabricated and field bolted to the fourth floor timber girders and column, directly above the third floor column to be removed. Next, 2¾"-diameter tie rods were connected to each gusset plate and to the collar with pins and clevis. After the framing system was inspected, jacks were placed below the fourth floor on each side of the column to be removed. The four tie rods were tightened as load was slowly applied to the jacks. A surveying crew monitored floor elevations as jacking proceeded. At this point, the tie rods were fully tightened, the jack pressure removed, and the load fully transferred prior to saw cutting the column from the third floor.

The north wall of the warehouse was in very bad condition, both structurally and cosmetically. After carefully considering various options, the conclusion was to com-

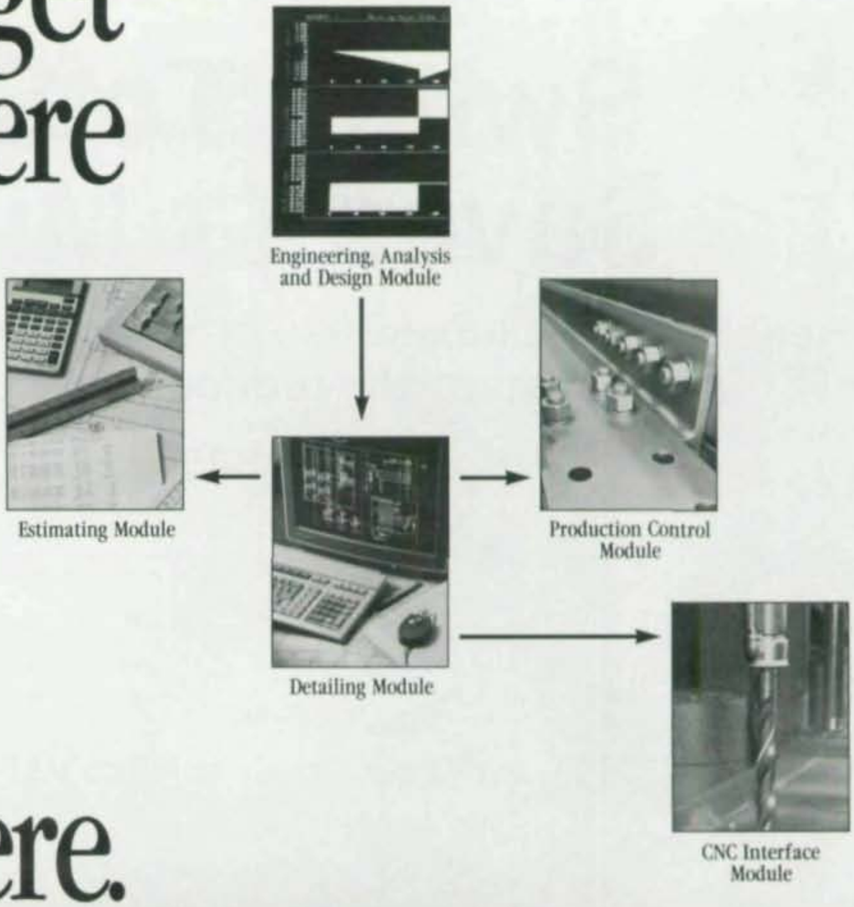
pletely remove the wall and replace it with a structural steel frame supporting brick veneer cladding. The new steel frame provided both gravity support for the 8-story structure and lateral support for the northern 76'-6" section of the warehouse.

Ticket windows at the first story prevented the use of diagonal bracing, and as a result the lateral wind forces are resisted by a rigid frame consisting of W16x45 columns on approximately 5"-8" centers connected to a horizontal W16x45 member with welded moment connections. Above the second floor, diagonal bracing consisting of TS6x4x¼ tubes were located between window openings. The diagonal tubes were located within the steel stud wall.

During the demolition of the brick wall, the existing wood framing was shored to the ground and laterally braced with steel cables. The wood floor joists were then connected to the new steel beams with continuous steel bearing plates and standard steel strap anchors.

Wm. Barton Wallis, P.E., is a vice president with Bliss & Nyitray, Inc., a consulting engineering firm headquartered in Miami.

How to get from here



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Switch To Steel Saves $1 Million

Redesigning a mixed-use project speeded construction and substantially reduced construction costs

When John Ciardullo—a practicing architect with formal training and practical experience in structural engineering—first saw the plans for a new multi-use building in Manhattan, he knew a redesign would save money.

The 22-story mixed-use building features retail on the first two stories, two gymnasiums—one stacked on top of the other—for a nearby prestigious private high school on the middle five floors, and residential space on the upper fifteen floors. As with so many residential projects, the original designer automatically specified concrete without pricing a steel alternative.

Fortunately, Ciardullo, principal of John Ciardullo Associates in New York City, was in the developer's office on other business and noticed the plans. Ciardullo has an undergraduate degree in civil engineering and worked as a structural engineer for five years before getting advanced degrees in architecture. "What I can do that most other architects can't is to redesign structural plans," he explained.

Because New York City code requires setbacks above 150' and because the two stacked gyms required large column-free areas, Ciardullo realized that steel might be a more economical approach for the project. The original design included massive concrete transfer beams, and Ciardullo was convinced that steel could do the job with less bulk and cost. With the owner's permission, he contacted the Steel Institute of New York, which in turn referred the problem to AISC Marketing, Inc., in Pittsburgh.

Within two weeks, AISC Marketing sent Ciardullo a preliminary design study. Included were steel requirements as well as several sketches showing proposed beam, column and bracing member framing. Based on AISC Marketing's proposal and Ciardullo's calculations, the owner agreed to a steel redesign.

The redesigned structure consists of flat slab concrete construction on the two bottom retail floors with steel framing above. On the third to eighth floors, a steel frame with a 2" metal deck was used. The project's engineer, Andrew Gyimesi of New Hope, PA, designed composite construction through the eighth floor to ensure the smallest member sizes. Spray-on fireproofing was used on the lower floors. Altogether, the project used 1,400 tons of steel.

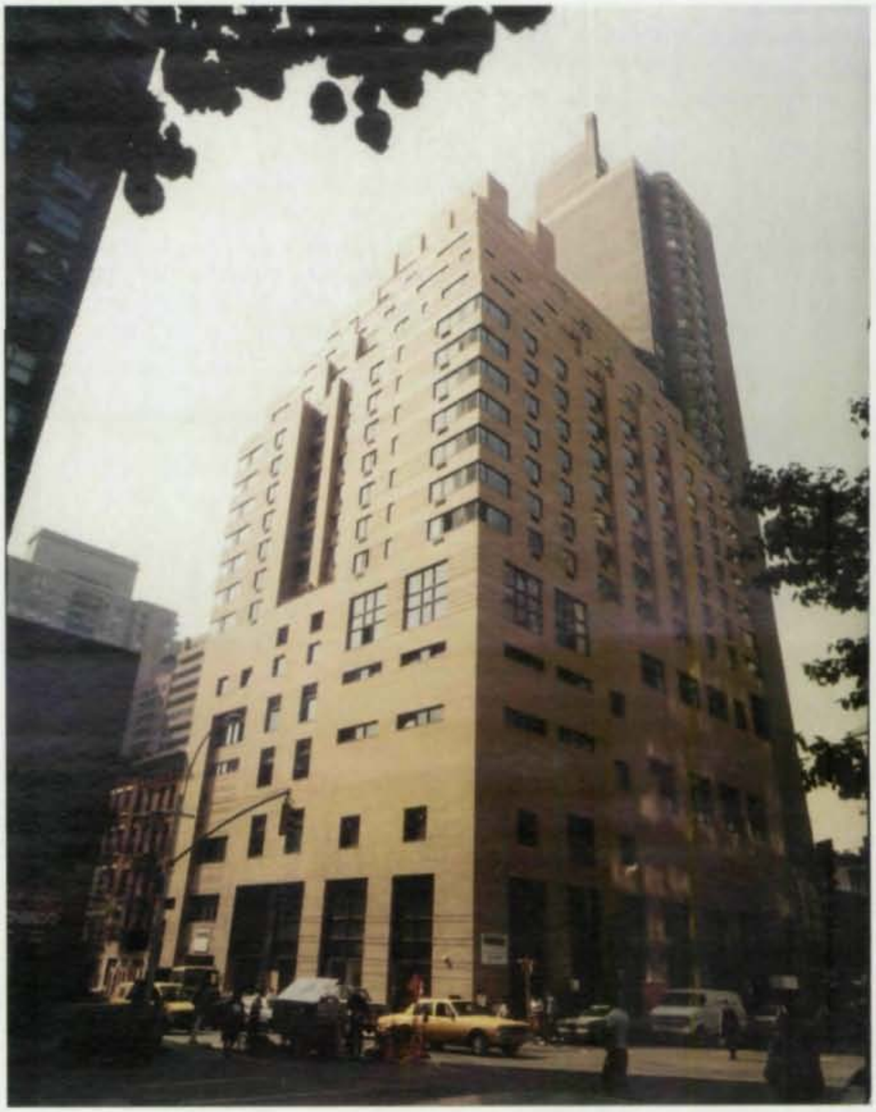
For the most part, the lower floors are designed for gravity loads. "The wind in the short direction is taken up by an internal wall with diagonal bracing, while the wind in the long direction on the lower floors is taken up by diagonal bracing up to the eighth floor in an exterior wall abutting a neighboring building," Ciardullo explained.

The framing for floors three through eight were complicated by the two column-free gyms. On the eighth floor, three 70′ long, 98" deep plate girders support the load of the residential floors above. "We started by designing a truss, but after meeting with the fabricator, we determined it would be less expensive to use a plate girder," Gyimesi explained. These steel girders, with 5" x 33" flanges are a far cry from the 10′ wide by 9′ deep reinforced concrete girders specified in the original design. At the fifth floor, six plate girders support the load of the gym above.

Due to their large size, great care was needed for the delivery and erection of the plate girders. "The city allows us to close one lane on the adjacent road for our crane," explained Don Whittemore, project manager with AISC-member Canron Construction Corp., the project's fabricator and erector. "In addition, they allow us to selectively close a second lane with prior approval." The largest members were trucked in—one at a time in a carefully sequenced process—early on a Saturday morning. As erection began on one, the next would arrive for off-loading.

Above the eighth floor, the steel frames support prestressed concrete planks—a construction method not used in New York City since the 1930s. "We considered a concrete slab, but a plank system was much more economical," Ciardullo explained.

The 8"-thick by 8′-wide precast planks are prestressed and therefore could not be welded at the bottom. Instead, the planks have



In addition to reducing the construction cost, the steel frame simplified the design and construction of the building's code-required setbacks. Photography by Jack Brennan embedded top plates that are welded to the columns, Gyimesi said. "Of course, only the planks that touch columns are welded to them. The rest of the planks are welded to each other at the $1/3$ and $2/3$ points of their lengths." By tieing the planks together, the design provides a diaphragm to transfer loads. Fireproofing on the upper floors is provided by sheetrock.

The residential apartments range in sized from studio efficiencies to luxurious four-bedroom units. The design and layout of each unit is effected by its location within the building.

This project was one of the first designed under a new zoning code requiring setbacks on projects over 150'. Therefore, every second level above the fifteenth floor was recessed—which created a terrace on every other floor.

Each setback required a load transfer, which, of course, was easier to manage with steel rather than concrete. For this project, the transfers were handled by slightly increasing the beam size in the east-west direction.

Substantial Savings

To save time, the contractor opted to have the beams and columns fabricated as one piece to further speed erection. General contractor on the project was Marson Construction Co., Bronx, NY.

These "tree columns" resemble double crosses and their use substantially speeds erection by transferring work to the shop where controlled conditions make connections simpler and faster. The tree columns, in turn, were bolted together at the center of the beams, which allowed them to act as a huge Vierendeel truss.

The exterior of the building is clad in brick, with strip windows to break up the vertical massing of the building and create horizontal movement. This movement is interrupted by large vertical insets at the center of both street-facing facades. Overall, the brick detailing and window arrangements—combined with the recessed terraces, accentuates the apartment arrangements and creates a notable presence for the building.

Ciardullo estimates that switching from concrete to steel reduced the cost of the $23 million project by $1 million. In addition, the weight of the structure was reduced by 30%.

Bearing Pad Design

Voss Engineering Inc. has published a new structural bearing pad design handbook. The handbook descirbes the performance characteristics of Sorbtex and Fiberlast for thicknesses 1" and greater. Also included in the publication is a description of the performance of Voss slide bearings. The handbook is the result of an extensive tesitng program and includes test results as well as design equations and tables. This data allows the design engineer, architect and detailer an opportunity to make design decisions based on material performance.

A bearing pad design computer program also is available. The program allows the user to design Sorbtex, Fiberlas and Voss slide bearings using an IBM PC or compatible computer.

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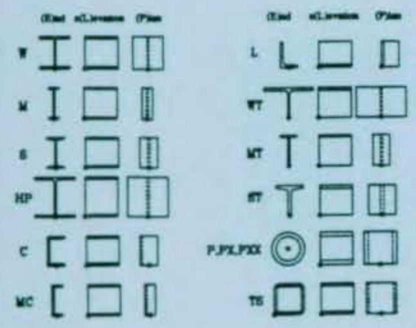

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