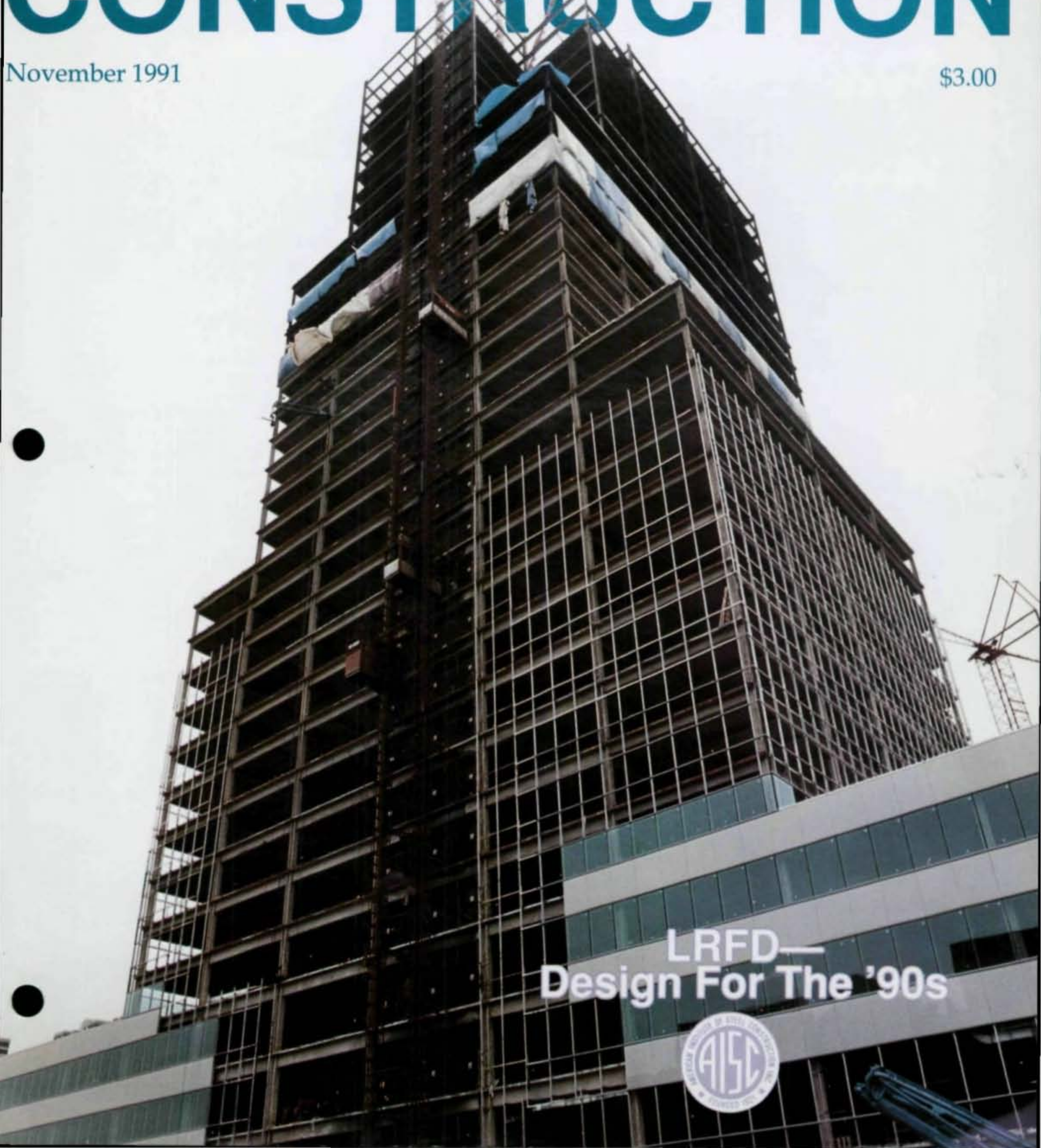
MODERN STEEL CONSTRUCTION

November 1991 $3.00



LRFD—
Design For The '90s

UNITED STEEL DECK, INC. DECK DESIGN DATA SHEET No. 13

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PRODUCTS	GALV. G90 or G60 I.	PHOS/PTD II.	PRIME PTD III.	GALV. + PAINT IV.	FINISH PAINT V.	UNCOATED VI.
1½" & 3" Roof Decks	X		X	X	X	
4½", 6", 7½" Roof Deck	X					
1½", 2", 3" Composite Deck	X	X				
UFX36, UF2X, UFS (Form Deck)	X			X	X	X

THE TABLE REPRESENTS NORMAL INVENTORIES; HOWEVER ANY FINISH ON ANY PRODUCT MAY BE AVAILABLE ON SPECIAL ORDER.

NOTES — ROMAN NUMERALS IN THE TABLE CORRESPOND TO NUMERALS IN NOTES.

I. A. CHECK U.L. FIRE RESISTANCE DIRECTORY FOR FINISH REQUIREMENTS. GALVANIZED DECK SHOULD BE USED ON *ROOF* CONSTRUCTION WITH SPRAYED FIRE RESISTIVE MATERIALS. (SFRM).
B. GALVANIZED DECK IS RECOMMENDED FOR HIGH HUMIDITY AREAS.
C. GALVANIZED ROOF DECK IS RECOMMENDED FOR ROOF CONSTRUCTIONS WITH INSULATION BOARDS THAT ARE FASTENED TO THE DECK WITH PIERCING FASTENERS.
D. USD RECOMMENDS THE USE OF GALVANIZED MATERIALS FOR MOST EXPOSURES.
E. GALVANIZED STEEL IS COVERED BY ASTM A446; GALVANIZING IS COVERED BY ASTM A525; G60 AND G90 ARE COATING WEIGHTS.

II. A. "PHOS/PTD." MEANS THE FLOOR DECK IS ONLY PAINTED ON THE EXPOSED SIDE—THE CONCRETE SIDE SHOULD DEVELOP TIGHT RUST BEFORE THE CONCRETE IS POURED.
B. USE ONLY FOR INTERIOR APPLICATIONS—I.E. OFFICES OR HOTELS.
C. CHECK U.L. FIRE RESISTANCE DIRECTORY—SEE NOTE I.A.
D. "PHOS./PTD." IS APPLIED TO ASTM A611 STEEL.

III. A. "PRIME PAINTED" MEANS A PRIMER COAT OF PAINT IS APPLIED OVER CLEAN BARE STEEL. THE PRIMER PAINT IS FORMULATED TO HAVE "TOOTH" TO HOLD SUBSEQUENT APPLICATIONS OF FINISH PAINT BUT IT IS NOT INTENDED TO PROVIDE EXTENSIVE WEATHER PROTECTION; IT IS FREQUENTLY LEFT EXPOSED IN WAREHOUSES AND MANUFACTURING PLANTS, AND WHEN USED WITH SUSPENDED CEILINGS.
B. USE FOR BALLASTED ROOFS OR ADHERED ROOF SYSTEMS— SEE NOTE I.C.
C. SALT SPRAY (AND OTHER) TEST RESULTS ARE AVAILABLE ON REQUEST.
D. "PRIME PAINTED" DECK IS MADE FROM ASTM A611 STEEL.

IV A. "GALV. + PAINT" MEANS PRIMER IS FACTORY APPLIED OVER GALVANIZED STEEL. THE PRIMER PAINT IS AS DESCRIBED IN III.
B. THIS FINISH IS MOST ECONOMICAL WHEN A FINAL COAT OF PAINT IS TO BE FIELD APPLIED.
C. USE IN HIGH HUMIDITY AREAS—THE PAINT PLUS GALVANIZING PROVIDES EXTREMELY GOOD MOISTURE PROTECTION.
D. "GALV. + PAINT" USES ASTM A446 STEEL.

V. A. FINISH COATS OF PAINT CAN BE FACTORY APPLIED. THIS IS DONE ON THE COILS OF STEEL BEFORE FORMING INTO DECK. ALMOST ANY COLOR OR PAINT TYPE CAN BE USED—HOWEVER TO BE ECONOMICAL, THE ORDER SHOULD BE FOR AT LEAST 20,000 SQUARE FEET.
B. WHEN INSTALLING DECK WITH A SPECIAL FINISH, SCREWED SIDE LAPS ARE RECOMMENDED: AND, IN MOST CASES, SCREWS, PNEUMATIC OR POWDER DRIVEN FASTENERS SHOULD BE USED AT SUPPORTS.
C. FINISH PAINT IS NORMALLY APPLIED OVER GALVANIZED STEEL CONFORMING TO ASTM A446.

VI. A. UNCOATED STEEL MEANS THERE IS NO COATING AT ALL. IT IS FREQUENTLY REFERRED TO AS "BLACK" STEEL.
B. UNCOATED STEEL CONFORMS TO ASTM A611.

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MODERN STEEL CONSTRUCTION

Volume 31, Number 11 November 1991

The 520'-tall Newport Office Tower in Jersey City, NJ, features a perimeter frame and braced core combination. In addition, outrigger trusses were provided at mid-height and at the top of the tower to stiffen the building and greatly improve its aerodynamic wind resistance. Photo by Sal Boccuti, Ambler, PA

FEATURES

NEWS AND DEPARTMENTS

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Editorial Staff

Scott Melnick,
Editor
Patrick M. Newman, P.E.,
Senior Technical Advisor
Cynthia J. Zahn,
Senior Technical Advisor

Editorial Offices

Modern Steel Construction
One East Wacker Dr.
Suite 3100
Chicago, IL 60601-2001
(312) 670-5407

Advertising Sales

Pattis-3M
7161 North Cicero
Lincolnwood, IL 60646
(708) 679-1100
FAX (708) 679-5926

A Better Mousetrap

I recently read an article describing various efforts to build an improved mousetrap. And in several cases, inventors had succeeded—yet, despite the well-known axiom, no one beat a path to their door.

For the last half of the 1980s, the authors of the Load and Resistance Factor Design Specification experienced the same abandonment. Sure a few of the more innovative East Coast firms started using LRFD. After all, LRFD is not only a more rational design method, but, in many cases, it also is more economical than ASD.

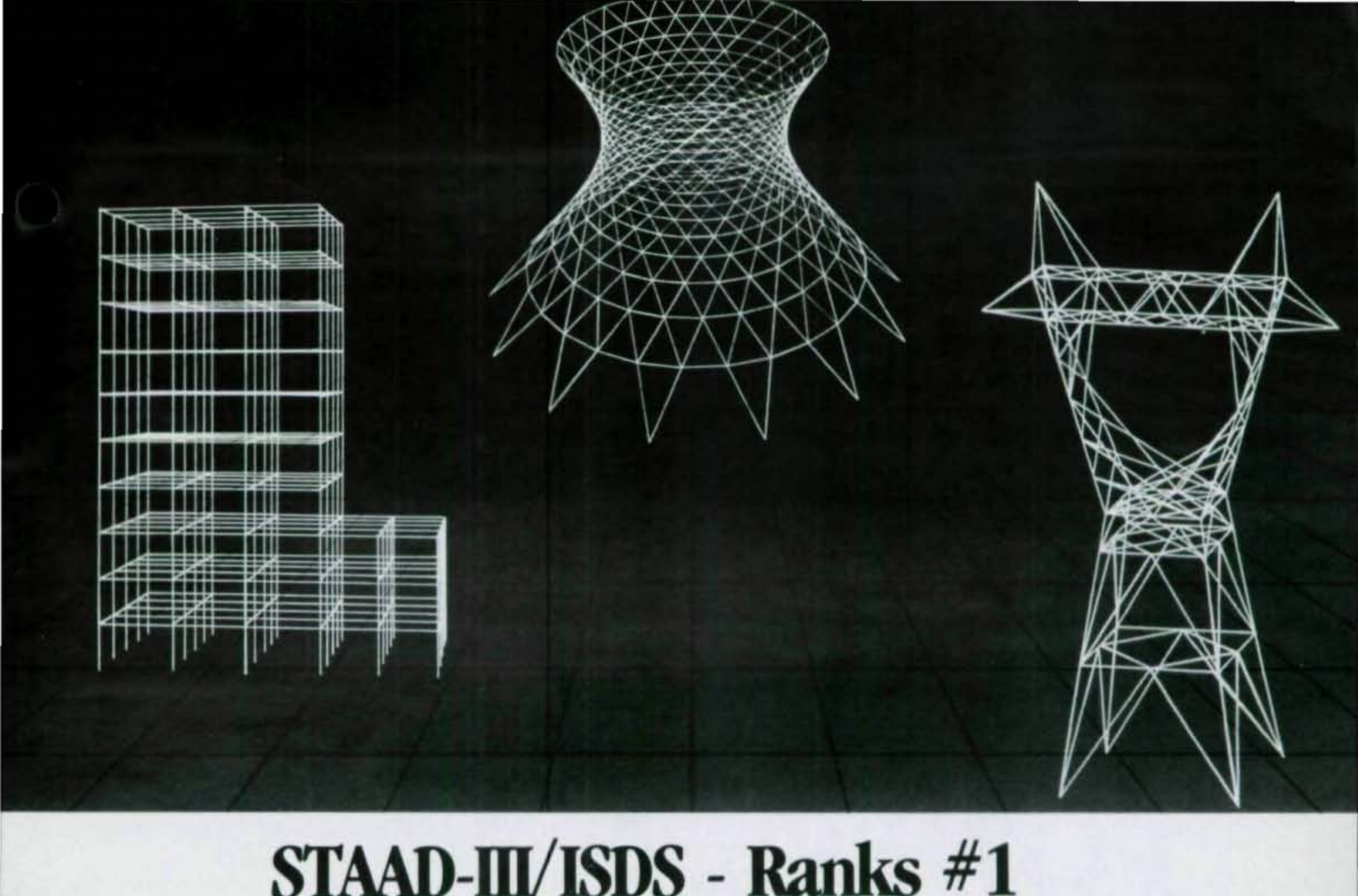
As Akbar Tamboli—a vice president at one of those firms that began using LRFD in 1986—explains in an article beginning on page 32: "There's a savings on most projects to the client and that gives us a competitive advantage."

How much of an advantage? In their article beginning on page 24, Tom Culp and Ravindra Mathur compare the design of typical office floor beams using LRFD and ASD and conclude that for most of the studied spans, the beam sizes obtained using LRFD with A36 steel are the same as the beam sizes using ASD with A572 Grade 50 steel. "Since the serviceability requirements for the two cases are the same, using A36 steel and LRFD design would always be more economical as compared to A572 Grade 50 steel and ASD design," they conclude.

And that can translate into direct savings. On the One Arizona Center project (pages 36-39), the engineer could design office live loading of 80 psf using LRFD for the same cost as 50 psf live loading with ASD.

These are compelling reasons for switching, and the move towards LRFD seems to be accelerating. Already, you would be hard-pressed to find a college engineering program in this country that does not teach LRFD (though some are still teaching ASD too). And even some firms on the West Coast are starting to use LRFD. "Implementing LRFD for steel design was considered professionally prudent," explained Californian William Andrews in an article beginning on page 12. "LRFD addresses strength and serviceability limit states in a more rational procedure and produces a more economical structure." In addition, Andrews discovered that the learning curve for using LRFD is very rapid.

Engineers still aren't stampeding to buy LRFD's better mousetrap; but the successful firms are steadily moving in that direction. **SM**



Correspondence

Dear Editor:

I have read your editorial, "False Initiative" in the September 1991 edition of *Modern Steel Construction*. I agree with your comments that the playing field for short-span bridges has been "tilted".

The Portland Cement Association began an intensive effort to offset the Timber Bridge Initiative introduced in the Senate by Senator Byrd (D-WV) in 1989, pointing out, as does your editorial, the existing structural deficiencies of timber bridges as listed in the National Bridge Inventory. Our efforts fell on deaf ears. Our concern was not with the cost of the program, which is authorized at $3 million, but with the precedent being set to subsidize a specific industry.

As the Senate timber bridge initiative was set in place, we realized the greater risk of perpetuating this program lay in the renewal of the Federal Aid Highway program, which is no before the Congress. Early in 1990, PCA undertook a survey of short-span bridges in all the states to determine bridge needs. In five states alone, we found more than 150,000 spans with a cost to replace or rehabilitate exceeding $500 million. Our guesstimate for a national replacement program would be in the range of $3-$5 billion. While these figures are not astronomical compared to other federal programs, they do represent a problem of bridge financing. Bridges under 20′ in length do not qualify for federal assistance under the highway program, and, according to the various state highway departments, these structures are either on a reduced schedule for maintenance, or none at all due to tight budges at the state and county levels.

At this juncture, PCA proposed to the Subcommittee on Surface Transportation, House Public Works and Transportation Committee that a "Short-Span Bridge Program" specifically for spans under 20′ in length be made part of the highway program to be reauthorized this year. The proposal included a demonstration program that would include alternative materials and designs in steel, concrete and wood. In short, the material to be selected would be left in the hands of the local officials to make more cost effective use of taxpayer dollars.

Our proposal called for a survey of these structures to learn the total needs of bridges on rural and minor arterial roads. These routes often serve as a starting point for the movement of commodities and agricultural products that travel the Interstate System. Further, the rural parts of our nation are merging with suburbia and these routes serve commuters to job sites as well as providing access to manufacturing and distribution system needs.

As the Senate and the House moved forward to draft highway legislation, we learned of Senate Majority Leader George J. Mitchell's plan to include a $20 million timber bridge initiative in the Senate version. While our discussions with Members and staff of the House Public Works and Transportation Committee appeared to be positive, we discovered, on release of the House proposal, of the inclusion of the timber bridge initiative similar to that of the Senate. However, the House increased the amount of funds to be authorized to $200 million over the five year life of the bill. Representative Bud Shuster (R-PA), ranking Republican on the House Surface Transportation Subcommittee (and a member of the so-called "Big Four" of the House Public Works and Transportation Committee), at our request, questioned the "pork" in this proposal. Although Mr. Shuster was unable to delete the timber bridge initiative from the House bill, he was successful in reducing the funding by half. Our industries owe thanks to Mr. Shuster.

Although we have made efforts to advise the steel industry of the timber problem, we note that it has been silent on this issue. If we are to "tilt" that playing field to a more even level for competing industries, there is still time for steel interests to write to members of Congress, for it is likely that a highway bill may not emerge until after Thanksgiving. Such letters should be targeted to members of the House Public Works and Transportation Committee objecting to a subsidy to timber at the expense of other materials.

The proliferation of timber for bridges does not stop at the congressional level. It appears that individual states are becoming involved in the issue in the hopes of garnering some of the $100 million to be provided. The speaker of the Virginia House of Delegates has requested the state department of transportation to develop a timber bridge initiative. Other states are sure to follow.

Fred Armstrong
Director, Government Services
Portland Cement Association

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Over 200 bridges in Ohio are constructed of weathering steel. Most of it produced by Bethlehem.

They use it for the same compelling reasons the New Jersey Turnpike Authority, Maine, Maryland, and numerous other states do.

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Ohio is particularly enthusiastic about the use of weathering steel for appropriate locations in conjunction with their jointless bridges.

They began the jointless concept in the 1930's. Since then, they've refined their jointless designs to the point where joints have been eliminated not just over the piers, but at the abutments, as well.

As a result, drainage problems associated with jointed bridges, such as failure of joint seals, or clogged drains overflowing onto structural members, have been eliminated.

Bridge lengths have also increased. Initially, the limit was 200 ft. Today, it's been increased to 300 ft., and even greater lengths are being constructed.

Engineers in the Buckeye state began using weathering

Ohio save a lot of money.
of paint.

steel in the early 1960's.

Low initial cost and minimum maintenance requirements make it highly competitive with other grades of steel. Also, it's attractive and blends in with its natural surroundings.

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You'll discover how we can save you a lot of money. Not to mention a lot of paint.

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West Coast Discovers LRFD

A firm's first experience with Load and Resistance Factor Design was simple and cost effective

The 67,000-sq.-ft. 2300 Camino Ramon office building was one of the last structure's built during California's booming construction period in the late 1980s and one of the first building's in the state to be designed using the Load & Resistance Factor Design Specification.

By William A. Andrews, C.E.

The last decade has witnessed a spectacular boom in new office construction in northern California's Diablo/San Ramon Valley. Strong population and job growth, along with high lease rates in San Francisco, increased the demand for new office space in this region.

Much of the office construction designed during the last decade was built without a specific tenant in mind. However, for the 2300 Camino Ramon building in the Bishop Ranch Business Park a single tenant—AT&T Network Systems—was signed to occupy the entire building prior to design and construction. As a result, the developer, PacTel Properties, paid special attention to the tenants needs during the design phase, including the strategic placement of electrified floor deck; carefully conceived column spacing; and special architectural features in the lobby.

Steel Structural System

A structural steel frame was selected for this three-story, 67,000-sq.-ft. building because of its space planning flexibility, economy, speed of erection, and proven earthquake performance. The building frame has bay sizes of 20′ x 20′, 30′ and 40′. The floor-to-floor height is 13′ on the upper floors and 14′ at the first floor. These heights were sufficient to allow mechanical ducts to pass through the ceiling space without penetrating the floor beams.

The floor system is 2½" of lightweight concrete fill over a 3" deep metal deck, in composite action with floor beams spaced 10' on center. Typically, the unshored composite beams were designed to support 80 psf live load. Floor vibration was investigated using the guidelines developed by Thomas Murray, Montague-Betts Professor of Structural Steel Design at Virginia Polytechnic Instittue and the winner of the 1991 AISC T.R. Higgins Lectureship award. (*AISC Engineering Journal*, 3rd quarter 1975 and 2nd quarter 1981; back copies of *EJ* articles can be ordered from: University Microfilm International, 300 North Zeeb Road, Ann Arbor, MI 48106, phone: 313-761-4700 ext. 533 or 534, fax: 313-665-7075. Murray's T.R. Higgins paper, "Building Floor Vibrations," will be published in *EJ*, 3rd quarter 1991. Copies can be obtained for $5.00 + $4.00 shipping from AISC, P.O. Box 806276, Chicago, IL 60680-4124 312-670-2400 ext. 433. One year subscriptions—four issues—cost $15; three year subscriptions—12 issues—cost $36.)

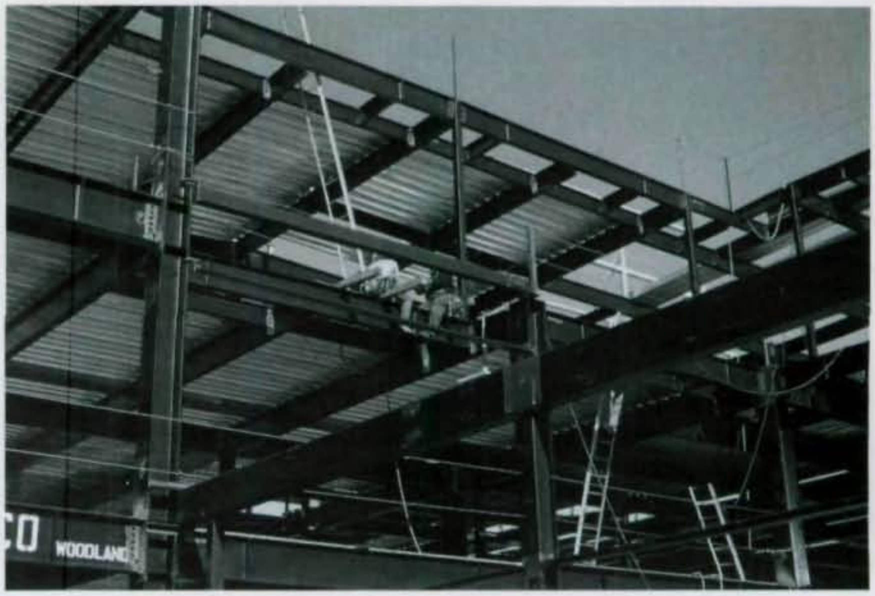
All of the floor beams are ASTM A36 steel. Electrified cellular deck was strategically located and blended with the standard floor deck to provide additional tenant flexibility.

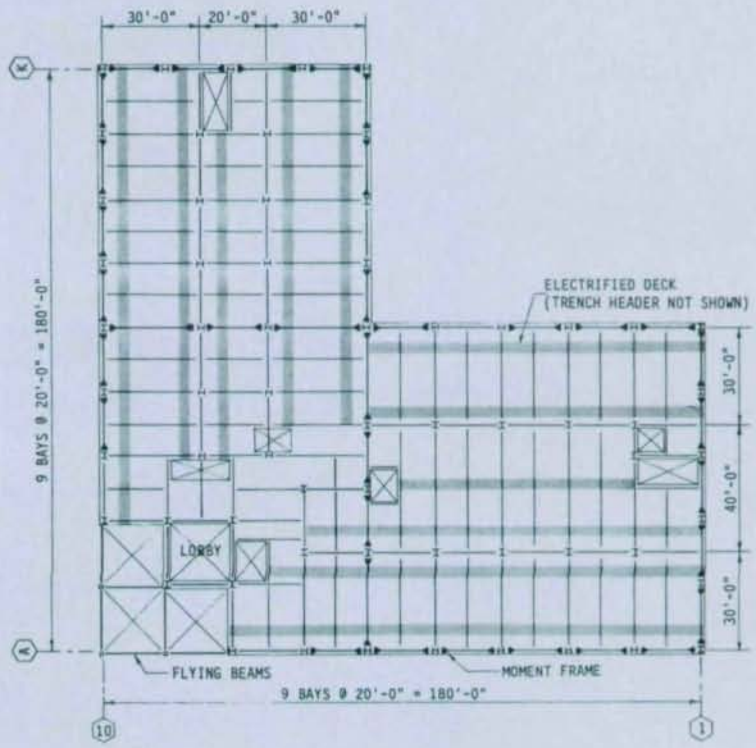
The foundation system consists of conventional spread footings and perimeter grade beams.

The exterior cladding consists of architectural glass fiber reinforced concrete (GFRC) panels and solar grey glazing. The main entry is a series of GFRC-clad "flying beams" leading into a two-story atrium lobby. The flying beams frame into columns but do not support floor structure, and therefore give the appearance of flying in space. The entry beams and columns are framed with ASTM A501 hollow structural sections. Architect on the project was Fee Munson Ebert, San Francisco. General contractor was L.E. Wentz Co., San Carlos, CA.

Switching To LRFD

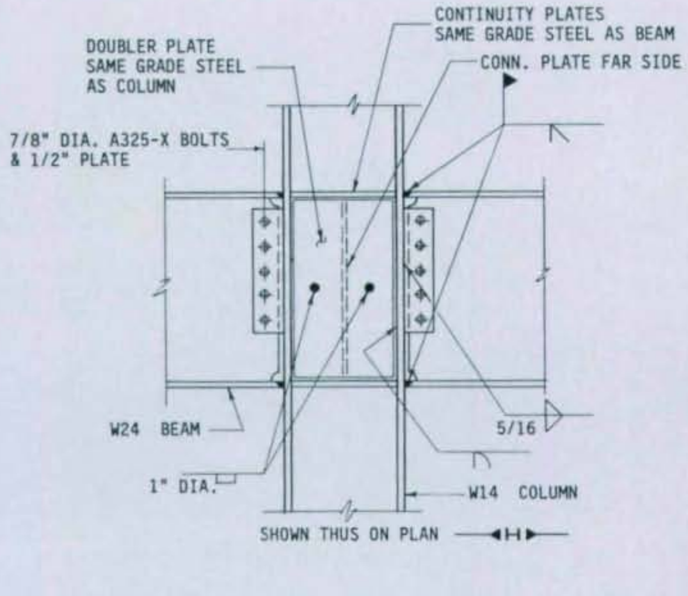
The 2300 Camino Ramon building is the first steel building designed by this firm using Load and Resistance Factor Design (LRFD).





2ND FLOOR FRAMING PLAN

Steel was chosen for this project because of its space planning flexibility, economy, speed of erection, and proven earthquake performance.



BEAM–COLUMN MOMENT CONNECTION

The building was designed with a Special Moment Resisting Frame (SMRF) to resist seismic forces without imposing architectural constraints.

During the design phase of the project, several factors influenced the decision to implement the LRFD Specification.

First, the firm's principals and staff had recently attended an AISC introduction to LRFD seminar.

Second, Ronald G. Vogel, the principle in charge of this project, was interested in utilizing the LRFD methodology in a steel design course he teaches at San Jose State University. Vogel is now in his third year of teaching LRFD at San Jose State.

And third, and most important, implementing LRFD for steel design was considered professionally prudent. LRFD addresses strength and serviceability limit states in a more rational procedure and produces a more economical structure. Using LRFD is consistent with the transition in the engineering community towards ultimate strength design methods for all materials.

Grasping the concept of LRFD was not difficult since ultimate strength concepts have been used by engineers for years. Learning to apply the LRFD Specification was accomplished as the different steel components were designed for the building. Structural elements designed using LRFD include: composite and noncomposite beams and girders; gravity columns; welded and bolted connections; and column base plates.

Substantial savings were realized in designing the composite floor and roof systems with LRFD. The composite floor beams and girders were designed by hand and verified using the composite beam tables in Part 4 of the LRFD Manual of Steel Construction. The LRFD procedure for composite beam design was found to be simpler and faster than the Allowable Stress Design (ASD) procedure. It also lends itself to a spreadsheet format, which, once developed, can be utilized on future projects.

The project used approximately 428 tons of structural steel.

Because of the lighter weight of steel members with LRFD, checking floor vibration levels is critical.

Only a minimum of "re-learning" was necessary to become proficient with LRFD, and making the switch resulted in a more efficient and effective design, while at the same time saving the owner construction time and dollars.

Seismic Design

The site is located in a seismically active region, 1½ miles from the Calaveras Fault, 10 miles from the Hayward Fault, and 28 miles from the San Andreas Fault. A moderate to severe earthquake on any of these faults could cause strong shaking of the site. A Special Moment Resisting Frame (SMRF) was chosen to resist these seismic lateral forces. The steel moment frame provides reliable resistance to seismic forces without the architectural constraints imposed by other systems.

During the design of this building, the LRFD seismic provisions were not yet published. Therefore, design of the ductile moment frame members were performed using ASD and the provisions of the 1988 Uniform Building Code for seismic zone 4. In the future, the firm will incorporate the *Seismic Provisions for Structural Steel Buildings—Load and Resistance Factor Design* ($5.00 + $4.00 shipping from: AISC, P.O. Box 806276, Chicago, IL 60680-4124 phone: 312-670-2400 ext. 433).

Moment frame columns are W14 sections, ASTM A572 Grade 50 steel. Moment frame girders are W24 sections, ASTM A36 steel. Beam-column moment connections are made with full penetration welds of the girder flanges to the columns and with fully-tightened A325-X bolted web connections for the girder gravity and seismic shear forces.

The column base connections are made with full penetrations welds of the column flanges to the base plates and with 1½" diameter A36 anchor bolts to the foundation. Concrete grade beams span from column-to-column to provide rotational stiffness for the column bases. The structure's irregular building plan creates significant torsional responses to earthquake ground motions, suggesting that a three-dimensional dynamic analysis of the lateral system be used. A response spectrum approach was employed using the 1988 UBC scaled normalized spectra.

The lateral system was modeled and analyzed using ETABS, a software package from Computers and Structures, Inc. The ETABS postprocessor, STEELER, was used to help size the moment frame members for stress and drift limitations. In the future, the firm expects to use the LRFD update of the program.

William A. Andrews, C.E., is a project engineer with the structural engineering firm of Meyer/Sumnicht Engineers (formerly Vogel & Meyer Structural Engineers), Walnut Creek, CA.

Design For The '90s

LRFD

Cutting Costs, Not Trees

While Merck & Co. desired an economic design, they also made sure the construction of their headquarters had a minimal impact on the environment

By Louis A. Occhicone, P.E., and Ramesh G. Keswani, P.E.

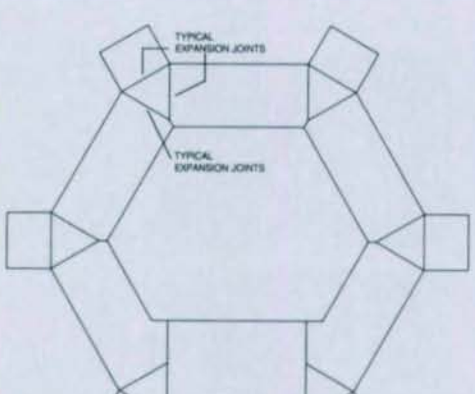
If you can visualize a hexagonal ring-shaped structure encircling five acres, you will begin to understand the scale of the Merck & Co., Inc., Headquarters building in Readington, NJ. And while the man-made portion is huge, it is dwarfed by the undeveloped portion of the site.

When Merck purchased the heavily wooded, 460-acre site for its headquarters, it made environmental preservation a priority. The master plan located the buildings away from the site's ponds and wetlands. When the project is complete, the building and roads will occupy only about 10% of the site, while 90% remains in its natural state. In addition, instead of simply bulldozing trees to make room for construction roads, 1,300 mature trees were transplanted to an onsite nursery. Two years later, the trees were replanted near the building and roads.

The project's architect, Kevin Roche John Dinkeloo & Associates, Hamden, CT, designed the building with three floors of offices to-

Merck & Co.'s hexagonal Headquarters building occupies five acres of a heavily wooded 460-acre site in Readington, NJ. The building's shape maximizes views without sacrificing employee movement between departments.

Photo by: Sal Boccuti, Ambler, PA.



Plan of Merck & Co. Headquarters
(building setbacks not shown for clarity)

taling 900,000-sq.-ft. atop a two-level, 700,000-sq.-ft. enclosed parking garage. Five segments of the hexagon conform to the typical framing, while the sixth, the main entrance segment on the south side of the hexagon, is different, primarily because of a 70'-high skylit atrium, two-story auditorium, a glass-enclosed cafeteria, a health-fitness center, and executive offices.

Visitors enter the Merck Headquarter's building through a five-story lobby atrium. The atrium is topped by a 108' x 60' rectangular skylight with 144 truncated glass pyramids rising from its surface. Supporting the skylight are triangular steel trusses custom designed to trace the shape of each pyramid.

Choosing LRFD

Since each beam, column and connection repeats itself throughout this huge structure, it was paramount that the typical structural elements were designed economically. To develop the most cost-effective framing scheme, the project's engineers, Weiskopf & Pickworth, New York City, considered a number of structural schemes before choosing a composite steel and metal deck system as the most economical. In addition, the engineers chose to use Load and Resistance Factor Design (LRFD) rather than Allowable Stress Design (ASD).

For all practical purposes, LRFD was in its infancy when this building went into design. AISC had only recently published the First Edition (1986) of the Specification, but the engineers at Weiskopf & Pickworth immediately recognized its significant cost saving potential as a more responsive design method than the traditional Allowable Stress Design.

By using different load factors for each load based on the predictability of that load and/or combination of loads, the engineers obtained a more reliable and cost-effective design. Material savings from each instance where LRFD called for less steel than ASD were multiplied as that element repeated itself again and again throughout this huge structure. LRFD had arrived just in time for this project.

ASTM A572 Grade 50 steel was selected over A36 steel or hybrid schemes combining the two strengths because the weight savings of Grade 50 steel more than made up for its higher unit cost compared with A36. All bolts used in the structure are high-strength ASTM A325 or A490 bolts.

Each side of the hexagon above the garage is 90'-wide by 300'-long.

The structure is designed as partially restrained (type PR) under the LRFD Specification. All members are designed to carry the factored gravity load as simple beams, while semi-rigid top and bottom angle connections provide lateral force resistance throughout the building, with the exception of the triangular components where braced frames are used.

Photos by Sal Boccuti

In the parking garage, which is shown in the bottom of this photo, 6" hollow core planks span to steel beams 18' on center. The use of LRFD in the garage portion of the building, where the live to dead load ratio is 0.55, resulted in a 12.2% weight savings for the steel beams.

Photo by Sal Boccuti

In the width of the building, there are typically three columns with two interior spans of 30' and two exterior cantilevers of 15'. The columns along the length of each side are 36' on center, generating typical bays of 36' x 30'. The main girders span 36' along the length of the side with filler beams at every 12' on center spanning 30' and cantilevering 15'. The columns in the lower and upper level parking area are prefabricated Firetrol columns. From the first floor to the roof level, columns are conventional with spray-on fireproofing.

The basic building is designed as partially restrained (type PR) under the LRFD Specification. All members are designed to carry the factored gravity load as simple beams, while semi-rigid top and bottom angle connections provide lateral force resistance throughout the building with the exception of the triangular components where braced frames are used.

The typical floor construction is a

two-hour fire rated composite slab consisting of 2½" of normal weight 3,000 psi concrete on a 3"-deep composite cellular 18 gage metal deck. The fire rating is achieved with cementitious spray-on fireproofing.

Beams, Girders And Columns

In a typical bay, all cantilevered beams spaced 12' on center are designed as non-composite. The live load to dead load ratio is 0.94. With LRFD, the beam sizes are at least one size less than if ASD had been used. An analysis of beams designed by LRFD vs. ASD showed an average savings of 0.42 lbs. per sq. ft. (6.8% savings). The savings could have been even larger, but the cantilevered beams at columns—which support the curtain wall—were designed for deflection and no savings were observed there. Steel fabricator on the project was AISC-member Interstate Iron Works Corp., Whitehouse, NJ.

All non-cantilevered beams are designed as composite beams with ¾"-diameter by 5"-long shear studs, except at places where large slab openings on both sides of beams are provided. In all the beams, the live load deflections are limited to L/360.

In the parking garage, 6" hollow core precast prestressed planks are used with a minimum 2" concrete topping. These planks span to steel beams 18' on center. Sloped steel framing combines with variable thickness concrete topping to provide drainage in the parking levels. The use of LRFD resulted in a weight savings of 0.39 lbs. per sq. ft. (12.2% savings) for the steel beams.

Because the LRFD Specification provides a live load factor of 1.6 compared to a dead load factor of 1.2, as the live load to dead load ratio increases, savings decrease. As a result, the savings are greater for the garage portion of the project, where the live to dead load ratio is 0.55, than for the office portion, where the live to dead load ratio is 0.94.

All of the steel columns are designed for gravity dead load, reduced live load and moments due to lateral load as per the BOCA Code. As a result, using LRFD did not reduce the steel weight compared to ASD. However, it still makes sense to use LRFD because using different load factors increases the predictability of load.

Expansion Joints

To handle thermal movements and stress, the building was divided into 18 separate structures by providing 1-½"-wide expansion joints at the junction of the structures. Each sixth of the hexagon is made up of a 90' x 300' rectangular area with setbacks, an equilateral triangular core, and a 60' x 90' node behind the triangular core.

Teflon coated slide bearing connections with stiffened seats placed at each expansion joint column link the structures into what appears to be one monolithic building. These connections also give each structure space to expand in summer and contract in winter. In total, the design called for 240 slide bearing connections.

Steel's Flexibility

The architects designed column-free offices on both the interior and exterior perimeters of the building. To accommodate the unobstructed offices, the outer bays were designed to cantilever off of columns set 15' within the structure. As the building features a glass and stone facade, these cantilevering perimeters needed to be carefully considered. Supporting the facade continuously around the perimeter of the building required provisions for the variation in deflection of the spandrel beams that are supported on cantilevers. To eliminate the problem of differential deflections, the facade was supported on steel trusses typically spanning 36' supported on floor beams cantilevering only from column lines.

The glass windows are 8'-long separated by 4'-stone sections. To shield the occupants from the sun, continuous sunshades circle the entire perimeter and courtyard of the building hexagon. These glass and aluminum sunshade panels cantilever off of perimeter steel beams that surround the building.

A glass-enclosed walkway wraps around the interior courtyard of the building at the upper parking level. The walkway enclosure is supported by architecturally exposed structural steel with each connection welded and ground smooth.

Each interior corner of the hexagon juts out into the interior courtyard; inside this corner, a three-story glass wall encloses a "communicating" stair. The stairs connect the three office floors and provide an unobstructed view of the woods in the courtyard. The stairs are constructed of steel tubes, channels and plates, all fabricated and erected to AESS standards.

On the first floor, the interior and exterior perimeters of the hexagon step back leaving terraces for planters and landscaping. The engineers easily accommodated the increased loads at these locations by supporting ribbed reinforced concrete slabs on steel beams with the metal deck acting only as a form.

With the main mechanical operations housed in a separate central utility plant building, mechanical rooms within the office building could be housed within the triangular cores at each corner of the hexagon. Each mechanical room services half the length of its adjacent building or 150' of the adjacent rectangular segments in each direction in the hexagon.

The headquarters, with its nearly 10,000 tons of steel, is scheduled to open in the spring of 1992 and will house 1,800 employees. Provisions for a future expansion were included in the master plan and include two more similarly-shaped buildings joined to the present structure.

Louis A. Occhicone is a partner and Ramesh G. Keswani is a project manager with Weiskopf & Pickworth, Inc., a New York City-based structural engineering firm.

#1 IN A SERIES

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Edward B. Finkel, P.E., Structural Engineer
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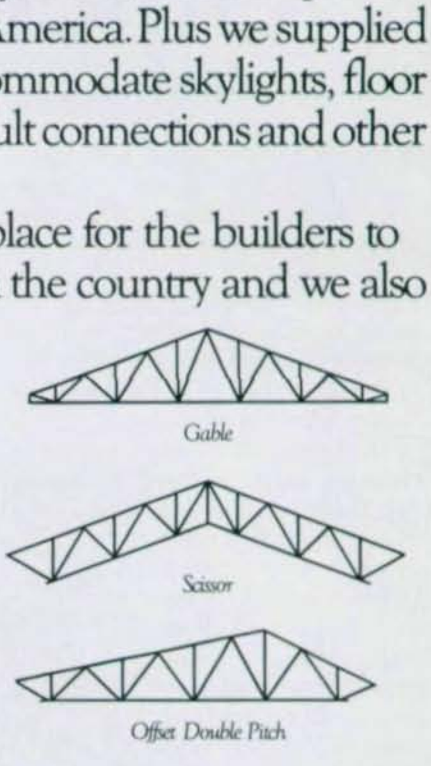

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LRFD Vs. ASD: A Comparison Of Office Floor Beams

By Tom Culp, S.E. and Ravindra Mathur, Ph.D.

While the Load and Resistance Factor Design (LRFD) Specification was issued way back in 1986, it is only now gaining an irrevocable foothold in the design community. Because of their familiarity with it, many engineers are still using the long-established Allowable Stress Design (ASD) method.

However, LRFD is based on an ultimate strength and reliability approach and is a more rational design procedure. With LRFD, the safety factors are comprised of load factors and strength reduction factors for different loadings, while the ASD method uses a safety factor on the yield stress value to obtain an allowable stress.

The use of Load and Resistance Factor Design usually reduces the beam sizes in steel buildings by one or two sizes, compared with Allowable Stress Design

A cursory examination shows that when design is based solely on strength considerations, LRFD is considerably more economical than ASD for typical office floor beams spanning 30′ to 46′. The question, though, is how much these savings are reduced by service related criteria such as deflections and vibrations.

During the process of switching to the LRFD method, Culp & Tanner undertook a study of serviceability issues. The loads and beam spans considered for this study are what would typically be encountered in the design of an office building floor. RAMSTEEL, a software program for automated design of steel structures from RAM Analysis, was used for the analysis and design of the steel members. The program has the option to design floor beams using ASD or LRFD with either A36 or A572 Grade 50 steel and also can check the vibration characteristics for a typical floor beam.

Design Specifications

The designs are based on the 1989 ASD Specification and the 1986 LRFD Specification.

A design live load of 80 psf was considered. The live load is reducible based on the UBC method. The allowable deflections are limited to L/360 for post composite live loads and L/240 for post composite superimposed loads.

The dead load, including slab, framing, ceiling, and miscellaneous load, is 49 psf for a 2" deck and 51 psf for a 3" deck. A dead load of 20 psf is added for partitions.

Design Examples

Four different cases were considered for the comparative study:

- ASD design using A36 steel for the typical beam (ASD-A36).
- LRFD design using A36 steel for the typical beam (LRFD-A36).
- ASD design using A572 Grade 50 steel for the typical beam (ASD-A572).
- LRFD design using A572 Grade 50 steel for the typical beam (LRFD-A572).

For each of the four cases the beam spans were varied from 30′ to 46′ with an increment of 2′. Center-to-center spacing between beams was selected as 8′, 10′ and 12′. The composite floor construction consists of 3¼" light-weight concrete over metal deck using ¾" diameter metal studs. A 2" deck is used for beams spaced 8′ on center and a 3" deck is used for beams spaced 10′ and 12′ on center.

Vibration Considerations

When using an optimized design method such as the one used by the RAMSTEEL program, it is important to carefully examine the vibration characteristics of the beams. It is a common perception that lighter sections may create vibration problems. As a result, this study carefully analyzes beam vibrations, and Tables 3 and 4 reveal that lighter sections often require less damping.

Two design criteria are evaluated. The first method recom-

Table 1: Optimally Designed Wide Flange Sections

span/ spacing	30'	32'	34'	36'	38'	40'	42'	44'	46'	
8'	16x31(13)	16x31(27)	18x35(20)	18x35(37)	18x40(33)	21x44(22)	21x44(38)	21x50(33)	21x50(54)	ASD-A36
	14x22(26)	16x26(21)	16x26(30)	16x31(29)	16x31(39)	18x35(32)	18x35(44)	18x40(42)	21x44(27)	LRFD-A36
	14x22(24)	16x26(14)	16x26(27)	16x31(21)	16x31(37)	18x35(26)	18x35(43)	18x40(35)	18x40(56)	ASD-A572
	12x19(24)	12x19(33)	14x22(28)	14x22(38)	16x26(29)	16x26(40)	16x31(35)	16x31(48)	18x35(34)	LRFD-A572
10'	16x31(30)	18x35(25)	18x40(23)	18x40(42)	21x44(30)	21x44(51)	21x50(45)	24x55(29)	24x55(46)	ASD-A36
	16x26(23)	16x26(31)	16x31(31)	18x35(27)	18x35(36)	18x40(36)	18x40(50)	21x44(36)	21x44(48)	LRFD-A36
	16x26(17)	16x26(31)	16x31(26)	16x31(47)	18x35(33)	18x35(57)	18x40(46)	21x44(30)	21x44(47)	ASD-A572
	12x19(35)	14x22(30)	16x26(23)	16x26(32)	16x26(45)	16x31(40)	16x31(57)	18x35(42)	18x35(59)	LRFD-A572
12'	18x35(30)	18x40(30)	21x44(23)	21x44(41)	21x50(37)	24x55(25)	24x55(40)	24x55(72)	24x62(60)	ASD-A36
	16x31(25)	16x31(36)	18x35(31)	18x35(46)	18x40(44)	21x44(32)	21x44(44)	21x44(62)	21x50(59)	LRFD-A36
	16x26(41)	16x31(33)	18x35(25)	18x35(46)	18x40(38)	18x40(70)	21x44(41)	21x44(73)	21x50(58)	ASD-A572
	14x22(35)	16x26(26)	16x26(38)	16x31(35)	16x31(51)	18x35(38)	18x35(55)	18x40(48)	18x40(68)	LRFD-A572

( ) indicates number of Nelson Shear Connectors

Table 2: Initial Load Deflection Before Composite Action

span/ spacing	30'	32'	34'	36'	38'	40'	42'	44'	46'	
8'	0.66 L/548	0.85 L/452	0.80 L/512	1.00 L/431	1.04 L/440	0.92 L/520	1.12 L/449	1.16 L/456	1.38 L/399	ASD-A36
	1.24 L/291	1.06 L/362	1.35 L/302	1.36 L/317	1.69 L/270	1.53 L/314	1.86 L/272	1.86 L/283	1.61 L/342	LRFD-A36
	1.24 L/291	1.06 L/362	1.35 L/302	1.36 L/317	1.69 L/270	1.53 L/314	1.86 L/272	1.86 L/283	2.23 L/248	ASD-A572
	1.90 L/190	2.45 L/157	2.04 L/200	2.57 L/168	2.11 L/216	2.59 L/186	2.52 L/200	3.04 L/174	2.67 L/207	LRFD-A572
10'	0.86 L/421	0.81 L/472	0.86 L/472	1.09 L/398	0.98 L/466	1.20 L/399	1.25 L/403	1.10 L/481	1.31 L/421	ASD-A36
	1.07 L/338	1.38 L/279	1.41 L/289	1.30 L/332	1.62 L/282	1.66 L/290	2.01 L/251	1.76 L/300	2.10 L/263	LRFD-A36
	1.07 L/338	1.38 L/279	1.41 L/289	1.77 L/244	1.62 L/282	1.99 L/242	2.01 L/251	1.76 L/300	2.10 L/263	ASD-A572
	2.47 L/146	2.09 L/184	1.76 L/232	2.21 L/196	2.74 L/166	2.70 L/178	3.28 L/154	2.91 L/182	3.47 L/159	LRFD-A572
12'	0.75 L/477	0.81 L/472	0.75 L/542	0.95 L/457	1.01 L/453	0.90 L/533	1.09 L/461	1.32 L/401	1.37 L/402	ASD-A36
	1.03 L/351	1.33 L/289	1.24 L/328	1.56 L/276	1.62 L/282	1.44 L/333	1.75 L/288	2.11 L/250	2.16 L/255	LRFD-A36
	1.28 L/282	1.33 L/289	1.24 L/328	1.56 L/276	1.62 L/282	1.99 L/242	1.75 L/288	2.11 L/250	2.16 L/255	ASD-A572
	1.93 L/186	1.65 L/232	2.11 L/194	2.13 L/203	2.64 L/173	2.38 L/201	2.90 L/174	2.91 L/182	3.47 L/159	LRFD-A572

Table 3: Perceptibility/Required Damping

(Without Partition Mass Included In Frequency Calculation)

span/ spacing	30'	32'	34'	36'	38'	40'	42'	44'	46'	
8'	LD/4.25	US/4.12	US/4.03	US/3.90	US/3.81	US/3.75	US/3.65	LS/3.58	LS/3.49	ASD-A36
	US/4.17	US/4.09	US/3.93	US/3.83	US/3.68	US/3.64	LS/3.53	LS/3.46	LS/3.45	LRFD-A36
	US/4.17	US/4.09	US/3.93	US/3.83	US/3.68	US/3.64	LS/3.53	LS/3.46	LS/3.36	ASD-A572
	US/4.08	US/3.87	US/3.81	US/3.64	US/3.63	LS/3.49	LS/3.43	LS/3.33	LS/3.31	LRFD-A572
10'	US/4.04	US/3.94	US/3.87	US/3.76	US/3.70	LS/3.60	LS/3.54	LS/3.51	LS/3.43	ASD-A36
	US/4.05	US/3.90	US/3.81	US/3.73	US/3.62	LS/3.55	LS/3.45	LS/3.43	LS/3.35	LRFD-A36
	US/4.05	US/3.90	US/3.81	US/3.67	US/3.62	LS/3.51	LS/3.45	LS/3.43	LS/3.35	ASD-A572
	US/3.89	US/3.82	US/3.75	US/3.63	US/3.50	LS/3.43	LS/3.33	LS/3.31	LS/3.22	LRFD-A36
12'	US/4.10	US/4.00	US/3.92	US/3.80	US/3.72	US/3.68	LS/3.59	LS/3.51	LS/3.44	ASD-A36
	US/4.08	US/3.94	US/3.86	US/3.73	US/3.64	LS/3.61	LS/3.50	LS/3.41	LS/3.35	LRFD-A36
	US/4.05	US/3.94	US/3.86	US/3.73	US/3.64	LS/3.53	LS/3.50	LS/3.41	LS/3.35	ASD-A572
	US/3.98	US/3.90	US/3.75	US/3.66	LS/3.52	LS/3.49	LS/3.56	LS/3.33	LS/3.24	LRFD-A572

Table 4: Perceptibility/Required Damping

(Including Partition Mass In Frequency Calculation)

Span/ Spacing	30'	32'	34'	36'	38'	40'	42'	44'	46'	
8'	US/3.94	US/3.82	US/3.75	US/3.64	LS/3.56	LS/3.52	LS/3.44	LS/3.38	LS/3.38	ASD-A36
	US/3.85	US/3.78	US/3.66	LS/3.57	LS/3.46	LS/3.42	LS/3.34	LS/3.28	LS/3.27	LRFD-A36
	US/3.85	US/3.78	US/3.66	LS/3.57	LS/3.46	LS/3.42	LS/3.34	LS/3.28	NP/3.20	ASD-A572
	US/3.77	LS/3.60	LS/3.56	LS/3.42	LS/3.41	LS/3.30	LS/3.26	NP/3.16	NP/3.15	LRFD-572
10'	US/3.78	US/3.69	LS/3.61	LS/3.52	LS/3.49	LS/3.41	LS/3.34	LS/3.33	LS/3.26	ASD-A36
	US/3.75	US/3.63	LS/3.56	LS/3.51	LS/3.40	LS/3.34	LS/3.26	LS/3.25	NP/3.19	LRFD-A36
	US/3.75	US/3.75	LS/3.56	LS/3.46	LS/3.40	LS/3.31	LS/3.26	LS/3.25	NP/3.19	ASD-A572
	US/3.64	LS/3.57	LS/3.52	LS/3.41	LS/3.30	LS/3.25	NP/3.17	NP/3.15	NP/3.09	LRFD-A572
12'	US/3.82	US/3.72	US/3.66	LS/3.58	LS/3.51	LS/3.47	LS/3.39	LS/3.31	LS/3.27	ASD-A36
	US/3.80	US/3.68	LS/3.60	LS/3.49	LS/3.42	LS/3.40	LS/3.32	LS/3.23	NP/3.19	LRFD-A36
	US/3.77	US/3.68	LS/3.60	LS/3.49	LS/3.42	LS/3.33	LS/3.32	LS/3.23	NP/3.19	ASD-A572
	US/3.70	US/3.64	LS/3.51	LS/3.44	LS/3.33	LS/3.30	LS/3.21	NP/3.17	NP/3.09	LRFD-A572

Perceptibility On The Modified Reiher-Meister Scale:
LD—Lower Half of Distinctly Perceptible Range
US—Upper Half of Slightly Perceptible Range
LS—Lower Half of Slightly Perceptible Range
NP—Not Perceptible

mended by Thomas Murray, Montague-Betts Professor of Structural Steel Design at Virginia Polytechnic Instittue and the winner of the 1991 AISC T.R. Higgins Lectureship award, calculates the damping characteristics of the beam based on its first natural frequency and amplitude of vibration and compares this damping with the actual damping provided by the floor system.

If the following inequality is satisfied, motion of floor system caused by normal human activity in office or residential environments will not be objectionable to the occupants:

$$D > 35\,A_0 f + 2.5$$

where D = damping in percent of critical, A_0 = maximum initial amplitude of floor system due to a heel drop excitation (in.), and f = first natural frequency of the floor system (hz).

(Murray's T.R. Higgins paper, "Building Floor Vibrations," will be published in AISC's *Engineering Journal*, 3rd quarter 1991. Copies can be obtained for \$5.00 + \$4.00 shipping from AISC, P.O. Box 806276, Chicago, IL 60680-4124 312-670-2400 ext. 433.)

The second design criteria plots the amplitude and frequency on the modified Reiher-Meister scale for various ranges of perceptibility.

Conclusions

Based on the analysis provided in this study, the following conclusions can be made for typical office floor beams spanning 30' to 46':

- Using LRFD results in beam sizes that are one or two sizes lighter than those obtained using ASD. This is true for A36 as well as A572 Grade 50 steel.
- An interesting phenomenon in Table 1 is that for most of the studied spans, the beam sizes obtained for LRFD-A36 and ASD-A572 are the same. Since the serviceability requirements for the two cases are the same, using A36 steel and LRFD design would always be more economical as compared to A572 Grade 50 steel and ASD design.
- The dead-to-live-load ratio in this study is 0.96 for a 3" deck with live loads of 80 psf. For designs using smaller live loads, a greater economy can be achieved when using LRFD as compared to ASD because of higher load factors for live loads in LRFD.
- In the case of LRFD-A572, the initial load deflections shown in Table 2 in all cases is greater than L/240. This may result in excessive cambers for the beams. Thus when sections are optimally designed by LRFD using A572 Grade 50 steel, the beam deflections, particularly initial deflections, should be carefully checked.
- As seen in Table 3, floor systems with beam spans ranging from 30' to 40' could experience vibration problems if the floor is free of partitions and the intended use is for a quiet environment. The required damping for this range is greater than 3.5% in all cases. However, beams over 40' in span typically require less than 3.5% damping, indicating no need for additional damping, even if the space is free of partitions.
- When partitions are present to provide additional damping to the floor system, beams fall within acceptable limits for the range of conditions covered in this study (Table 4).
- For all beams evaluated in this study, the Modified-Reiher-Meister scale indicates the floor system to be free of vibration problems as each beam is below the upper half of distinctly perceptible range. Thus, a considerable amount of damping from ceiling and partitions is assumed in this scale. Caution is recommended when using this scale for floor systems free of partitions and other means of damping.

Tom Culp, S.E., is a principal and Ravindra Mathur, Ph.D., is a project engineer with Culp & Tanner, Inc., a consulting structural engineering firm located in El Toro, CA.

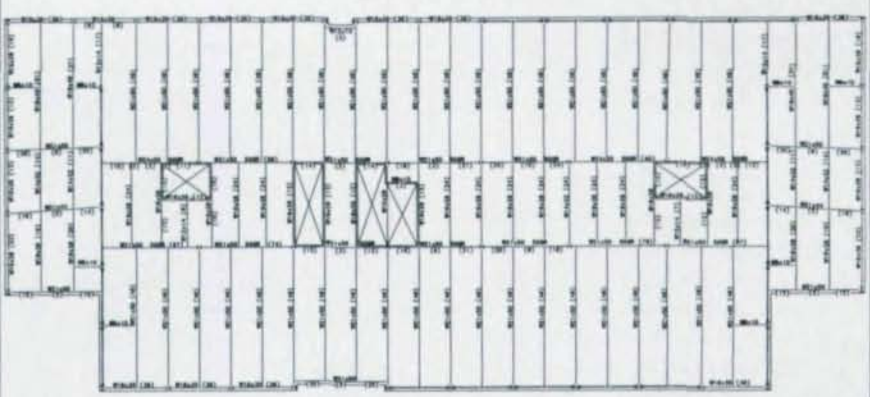
Switching To LRFD

By Tom Culp, S.E.

The well documented efficiencies of Load and Resistance Factor Design (LRFD) confirmed for our office that the switch to LRFD was a necessity. The question of when it would happen primarily was an issue of resolving two serviceability concerns—vibration and initial load deflection. The degree to which beam end restraints, construction techniques and other issues impact initial deflection is a matter of experience. Similarly, methods to calculate vibration perceptibility are available and well documented, but the designer must have a level of confidence, based on experience, that the system will perform well.

With LRFD designed beams being one or two sizes smaller than what was common with Allowable Stress Design (ASD), we found it necessary to limit initial deflection. By limiting the initial deflection, typical purlins were generally the same as our previous ASD design except A36 steel could be used in place of A572 Grade 50. Our initial deflection criteria generally did not constrain girder sizes. We expect that as experience is gained, we will be able to adjust our criteria and realize increased economies.

Choosing a good software package made the a actual transition from ASD to LRFD—at the production level—relatively simple. We are using RAMSTEEL, a design



The designers of this 10-story office building expect to reduce the weight of the steel framing on the upper floors by 5 lbs. per sq. ft. by using the LRFD Specification instead of the ASD Specification. This is the engineering firm's second experience with LRFD.

program for gravity steel framing which allows the user to graphically input the entire building. Once the model is input, full-floor comparative designs in both ASD and LRFD are as simple as changing one variable and rerunning the program. These comparative studies are extremely useful not only in tonnage comparisons, but also in gaining experience in the configuration and loading conditions.

(A listing of programs that incorporate the LRFD Specification is on page 30. If you know of other programs that incorporate LRFD, please contact: *MSC*, One East Wacker Dr., Suite 3100, Chicago, IL 60601-2001.)

Of course, there is always the concern, as with any program, that the engineer using it is not fully versed in the design or analysis procedures the program is performing. This, however, has not been a major obstacle in our office. With most schools now teaching LRFD, it is the design method of preference for most young designers. The fact that these recent college graduates perform the majority of beam member sizing in our firm has minimized the inefficiencies associated with re-educating our engineers.

Currently in design is a 10-story office building (31,000 sq. ft. per floor) over a 100,000-sq.-ft. basement. The basement roof framing extending past the footprint of the building supports plaza and traffic loads. The savings from LRFD design on the upper floors was mostly the cost premium between A36 and A572 for the typical purlins, which accounted for approximately 5 lbs. per sq. ft. of the total framing. At the plaza level, where the superimposed loads were much greater, serviceability issues did not typically impact member sizes. Savings at the plaza level were in excess of ¾ lbs. per sq. ft.

Software Programs That Support LRFD

Program	Company	Phone
AISC for AutoCAD	AISC, Inc.	312-670-5434
AISC Database		
CONXPRT		
WEBOPEN		
SCADA	American Computers & Engineers	213-820-8998
LRFD Database	Calpro	800-446-0959
M-STRUDL	C.A.S.T.	415-795-0509
CAST-UTILITY		
ETABS (STEELER)	Computers & Structures, Inc.	415-354-0234
AutoFLOOR		
SAP90[1]		
SD1C	ECOM Associates	414-354-0234
SD2C		
SD3C		
SD4C		
AutoSTEEL[1]	Engineering Design Automation	415-848-7080
Intergrated Beam Design[1]	Engineering Software Co.	817-861-2296
Multiframe Section Maker[2]	Graphic Magic, Inc.	408-464-1949
LRFDCOMP	Precision Programming	612-936-4031
RAMSTEEL	RAM Analysis	800-726-7789
STAAD-III	Research Engineers, Inc.	800-FOR-RESE
P-STEEL	Softek Services, Inc.	604-732-3763
P-FRAME(2D)		
P-FRAME(3D)		
StruCAD3D[3]	Structural Software, Inc.	713-984-9173

1 Available 4th Quarter 1991
2 MacIntosh Program
3 Available 2nd Quarter 1992

Design For The '90s LRFD

LRFD Saves $2 Million

The 37-story Newport Office Tower, one of the tallest buildings in New Jersey, was designed using the LRFD Specification. According to the engineer, using LRFD usually results in a savings. On this project, using LRFD saved about $2 million in steel costs, primarily by reducing beam and girder sizes.

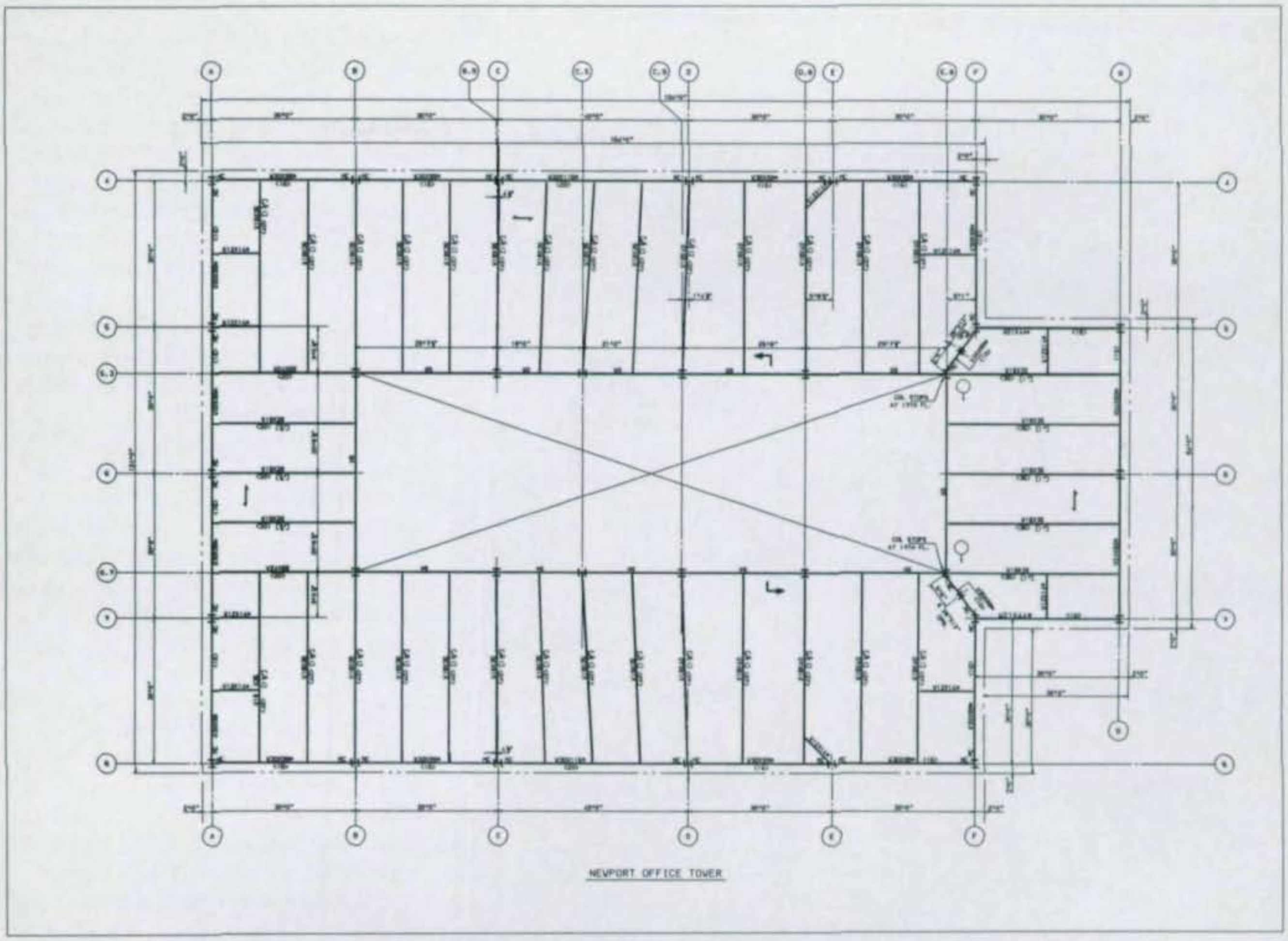
Since its introduction in 1986, LRFD has provided a competitive edge for this New York-based engineering firm

When the engineers for one of the largest office buildings in New Jersey began design, they chose a steel structural system both for its lighter weight and the ease with which modifications can be made.

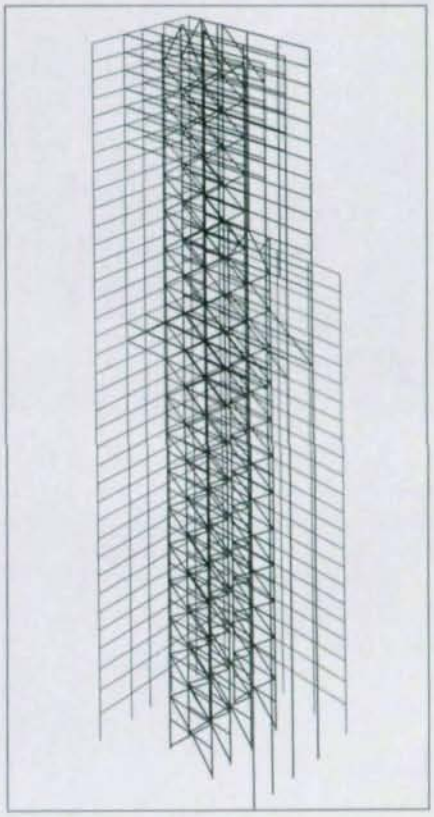
"The biggest factor on the Newport Office Tower project was the foundation conditions," explained Akbar Tamboli, P.E., a vice president with The Office of Irwin G. Cantor, P.C., in New York City. "The Jersey City-area has 100' of average fill over bedrock. A concrete-framed building would have substantially increased our foundation costs." To further reduce foundation costs on the 37-story, 1.1-million-sq.-ft. structure, the engineer opted to use slurry caissons instead of conventional concrete caissons. Geotechnical consultant on the project was STS Consultants, Ltd., Chicago.

Another important reason for choosing steel was the flexibility it provides, which is crucial in developer office buildings. "With steel, tenant changes and modifications are much, much easier. Without the ability to make inexpensive modifications, the owner, Melvin Simon Associates, would have had difficulty accommodating future tenant needs."

In addition, steel easily provided the long clear-spans desired by the



By combining a low-rise base with the tower portion of the building, the architect created a structure compatible with the surrounding community. In addition, the designers further diminished the scale of the building through the use of setbacks.



Outriggers, located at the 21st and 22nd floors and at the building's top help to control deflection and building sway. In addition, the outriggers reduced the weight of the structure by reducing the needed weight of core trusses and perimeter framing, which helped to reduce foundation costs.
Photo by Sal Boccuti, Ambler, PA.

owner. "We could have used joists with concrete construction, but that would have substantially increased the floor-to-floor height," Tamboli explained. Also, steel provided smaller column sizes. "Concrete columns become prohibitively large on a 37-story building." And the larger the columns, the less leasable space.

Just as the decision to use steel was a simple one, so to was the decision to use Load and Resistance Factor Design (LRFD).

"Our firm has been using LRFD since 1986," Tamboli explained. "Unless there's a code problem, we automatically use LRFD. There's a savings on most projects to the client and that gives us a competitive advantage." On the Newport Office Tower project, Tamboli estimates that the use of LRFD saved $2 million in steel costs, primarily by reducing beam and girder sizes.

When the firm first began using LRFD, they had to change a lot of software to accept the procedure. "But now that we're all set up for it, it takes no extra time for design."

On this project, the software included: Research Engineer's STAAD-III for column/beam design; McDonald Douglas' EASE program for wind analysis; AISC's CONXPRT for connections; and AISC's WEBOPEN to calculate web openings.

Structural Design

The 520'-tall building's structural wind system features a perimeter frame and braced core combination. In addition, outrigger trusses were provided at mid-height and at the top of the tower to stiffen the building and greatly improve its aerodynamic wind resistance.

"The main purpose of the outriggers was to control deflection and building sway," Tamboli said. "Secondarily, it reduced the weight of the structure by reducing the needed weight of core trusses and perimeter framing. And finally, the outriggers eliminated uplift on the foundation."

Because of the building's exposure to New York Harbor and its height, wind tunnel tests indicated that there might be wind loads of 90 psf on parts of the building—double the local codes requirement of 45 psf. The engineer received wind loading and criteria for building acceleration from RWDI in Guelph, Ontario, and designed the outriggers accordingly.

Outriggers Save 500 Tons

The mid-level outriggers, which occur at the 21st and 22nd floors, are composed of W14 x 192 steel members, while the outriggers at the top of the building are W14 x 176 steel members. Tamboli estimates that the use of the outriggers resulted in an approximate steel reduction of 500 tons. Steel fabricator was AISC-member Owen Steel Company.

"The disadvantage is that the outriggers create an obstruction on those floors, but the owner felt the dramatic cost savings more than made up for it."

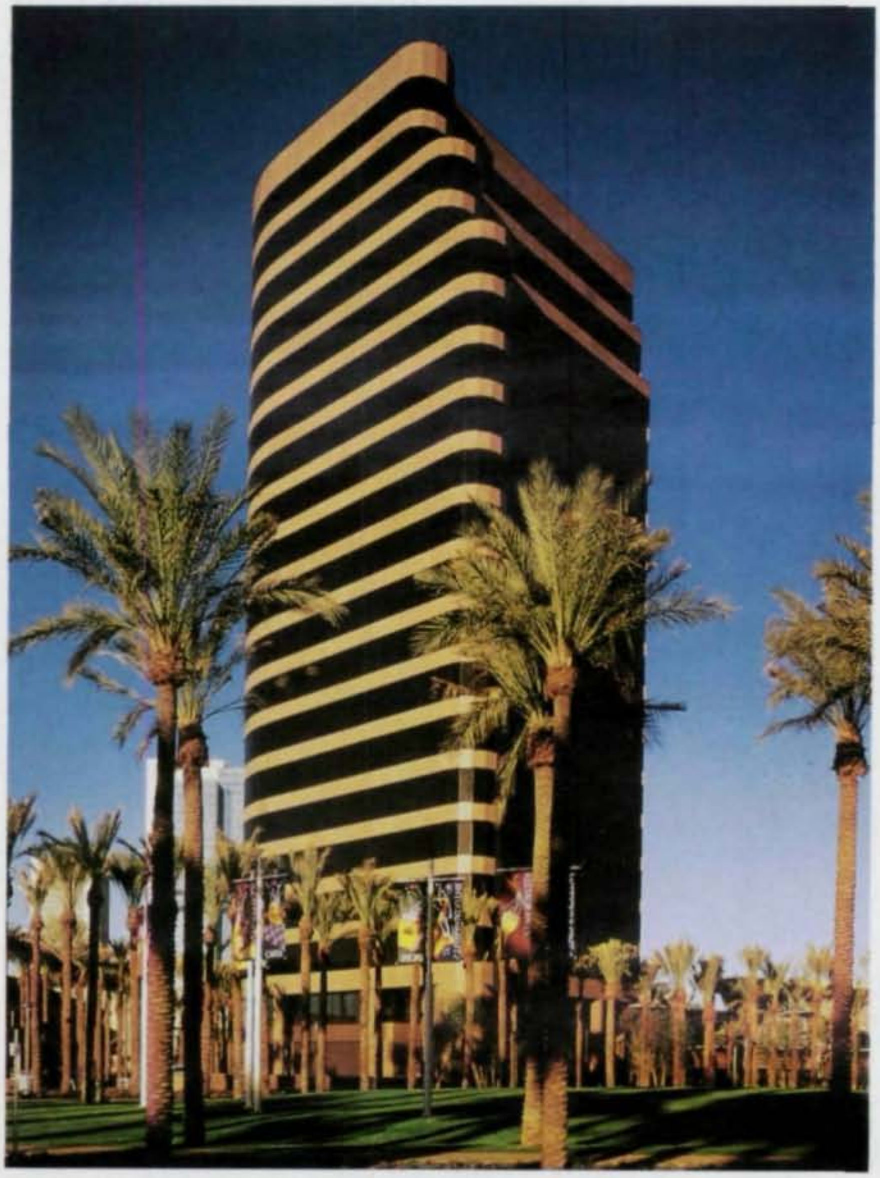
The building's columns, where governed by wind load, are A36 steel, while the columns governed by gravity loads are A572 Grade 50. The beams and girders are A572 Grade 50 steel, except for the perimeter wind girders, which are A36. "We didn't need higher strength steel in those areas where the moment of inertia was critical," Tamboli explained.

The perimeter column-beam connections are full-penetration field welded, while in the core connections are achieved with gusset plates welded to columns and beams field bolted to the gusset plates.

To create a column-free lobby, the designers sloped some of the columns through the third and fourth levels. The sloping members are 30' in length and are offset 6'. "We've been using this system for a number of years instead of transfer girders," Tamboli stated. "It reduces the needed quantity of steel and increases headroom."

Architect on the project was The Ehrenkrantz Group & Eckstut, New York City, with associate architects Emery Roth & Sons, also of New York City.

Steel Construction Meets Tight Deadline



Using LRFD instead of ASD is estimated to have saved 1.4 psf in overall steel tonnage on the One Arizona Center project in Phoenix. And according to the project's engineers, LRFD procedures are just as easily applied as ASD procedures.

One Arizona Center's steel frame was fully erected just 10 months after design began

By David A. Platten, P.E.

Tenant move-in commitments dictated a 10-month schedule from the start of design to the completion of the structural frame for One Arizona Center, a 360,000-sq. ft. office building in Phoenix. The 20-story building, which sits atop two-levels of below-grade parking, is the second in a series of three planned office buildings in the 18.5-acre, Arizona Center mixed-use project. Project architect was HKS, Inc., Dallas.

To meet the tight schedule, the engineer, Walter P. Moore and Associates, Inc., integrated a concrete below-grade structural system with a structural steel system above-grade.

Once structural steel was selected for the structural frame, value engineering efforts were focused on three considerations: Allowable Stress Design (ASD) vs. Load and Resistance Factor Design (LRFD); composite beam framing utilizing normal weight concrete vs. light-weight concrete; and design office live loading of code minimum 50 psf vs. 80 psf.

Composite Floor Framing

The composite floor framing system utilized for the building consisted of A572 Grade 50 steel beams at 10′ centers with 2", 19-gage metal deck plus 2½" of normal weight concrete. The slab was cementitous spray fireproofed to achieve the required fire rating.

In arriving at the selected system, four designs were considered:

- Design 1. LRFD using A572 Grade 50 steel beams at 10′ or 7′-6" centers with a 5¼" lightweight concrete slab.
- Design 2. LRFD using A572 Grade 50 steel beams at 10′ or 7′-6" centers with a 4½" normal weight concrete slab.
- Design 3. ASD using A572 Grade 50 steel beams at 10′ or 7′-6" on centers with a 5¼" lightweight concrete slab.
- Design 4. ASD using A572 Grade 50 steel beams at 10′ or 7′-6" centers with a 4½" normal weight concrete slab.

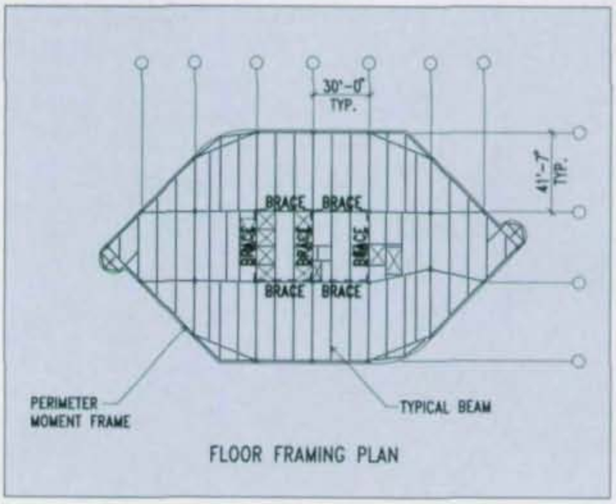
Typical beam spans were 41′-7", while typical girder spans were 30′. Each of the four designs was studied first assuming 50 psf live loading and then 80 psf live loading.

Designs 1. and 3. were eliminated as a result of limited availability of lightweight aggregate in the Phoenix area. A study of Designs 2. and 4. showed that the LRFD design was more economical than the ASD design. Furthermore, it was determined that the typical floor beam size and number of shear connections would be the same for an 80 psf live loading using LRFD and a 50 psf live loading using ASD. Essentially, the owner was able to offset the premium for the 80 psf live loading he desired by utilizing LRFD in lieu of ASD. Total floor steel weight was 5.6 psf for LRFD vs. 6.4 psf for ASD, a savings of 13%.

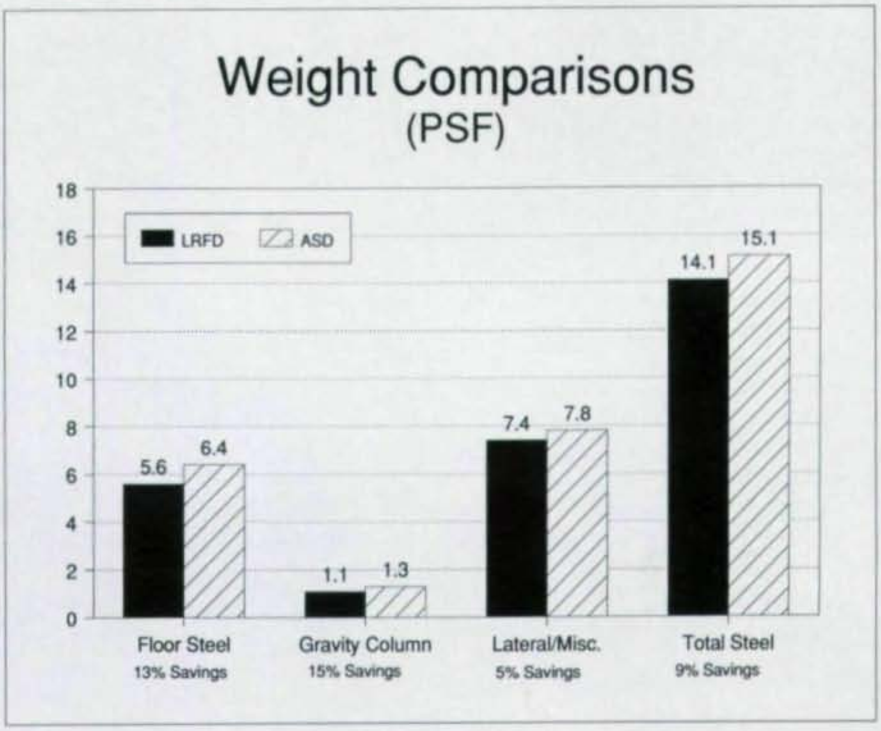
This was a substantial advantage to the developer, who wished to provide the higher floor live load for added tenant flexibility for computer rooms and filing areas.

Lateral Framing System

To meet the seismic requirements of the Phoenix Construction Code, a lateral framing system consisting of core K-braced frames



The project was designed using A572 Grade 50 steel beams at 10′ or 7′-6" on centers with a 4½" normal weight concrete slab. The lateral bracing consists of K-braced frames in combination with a perimeter moment frame.



Load And Resistance Factor Design (LRFD)

Slab Thickness	Beam Spacing	50 PSF Live Load		80 PSF Live Load	
		Typ. Beam Size	PSF	Typ. Beam Size	PSF
5¼"	10'-0"	W18x35	3.50	W18x40	4.00
5¼"	7'-6"	w16X26	3.47	W16X31	4.13
4½"	10'-0"	W18X35	3.50	W18X40	4.00
4½"	7'-6"	W16X26	3.47	W18X35	4.67

Allowable Stress Design (ASD)

Slab Thickness	Beam Spacing	50 PSF Live Load		80 PSF Live Load	
		Typ. Beam Size	PSF	Typ. Beam Size	PSF
5¼"	10'-0"	W18X40	4.00	W18X50	5.00
5¼"	7'-6"	W18X35	4.67	W18X40	5.33
4½"	10'-0"	W18X40	4.00	W18X50	5.00
4½"	7'-6"	W18X35	4.67	W18X40	5.33

in combination with a perimeter welded moment frame was selected. Use of the perimeter frame also provided excellent live load deflection control at building cladding support points. General contractor on the project was HuntCor, Inc., Phoenix.

The core K-braced frames consist of A572 Grade 50 steel beams and columns, as their design was controlled by strength requirements. Double angle brace members were designed for serviceability, and therefore A36 steel sections were selected.

Similarly, the perimeter welded moment frame consisted of A572 Grade 50 steel columns and A36 beams. As a result of a significant portion of the lateral framing system being controlled by serviceability, the difference in steel weight between LRFD and ASD was nominal. Lateral and miscellaneous framing weight was 7.4 psf for LRFD vs. 7.8 psf for ASD, a savings of 5%.

Steel fabricator on the project was AISC-member Zimmerman Metals, Inc., Denver. Steel erector was LPR Construction Co., Loveland, CO.

Gravity Columns

From a percentage standpoint, the greatest difference in steel weight between LRFD and ASD was observed in the design of the gravity columns. Because strength was the governing factor, A572 Grade 50 gravity columns were utilized. These columns weighed 1.1 psf for LRFD vs. 1.3 psf for ASD, a savings of 15%.

LRFD design is estimated to have saved 1.4 psf in overall steel tonnage, or about 9%, on this project. Assuming a unit price for structural steel of $0.25/lb., this translates into a savings of $126,000 for the Arizona Center's developer, Rouse-Phoenix Development Corp., Phoenix.

In addition to its cost saving benefits, LRFD is a more rational design procedure than ASD. It involves explicit consideration of limit states, multiple load factors and resistance factors, and implicit probabilistic determination of reliability. Also, our firm's experience has been that LRFD procedures are just as easily applied as are ASD procedures. While this wasn't the first LRFD project for Walter P. Moore, it was the first for company's Dallas office and its first use in Phoenix.

David A. Platten, P.E., is vice president and head of the Dallas/Ft. Worth Division of Walter P. Moore and Associates, Inc. He is a registered professional engineer in Arizona, Texas and Virginia, and a member of ASCE and the Consulting Engineers Council of Texas.

For One Arizona Center, the typical floor beam size and number of shear connections would be same for an 80 psf live loading using LRFD and a 50 psf live loading using ASD.

Bridge Rehabilitation

Lubrite self-lubricating bearings accommodate expansion, contraction and rotation of structural members—*without maintenance or* supplementary lubrication. The bearings also are unaffected by temperature extremes, immersion and/or corrosion. The company produces bearings for a wide variety of applications: flat expansion plates are designed to accommodate expansion and contraction in a single plane; radius plates are flat on one face and either concave or convex on the opposite face, with the radius plate accommodating the deflection or rotation of the structural member and the flat face providing for linear expansion and contraction; and spherical plates provide for rotation or deflection in any direction as well as normal expansion and contraction. All bridge bearings can be supplied in standard Lubrite or Lubrite F 100% teflon fiber mat, which offers a very low 0.03 coefficient of friction. The company offers a wide variety of completed assemblies, as well as design assistance. Also, quick turnaround is provided, which is particularly helpful in rehabilitation work.

For more information, contact: *Merriman, 100 Industrial Park* Road, Hingham, MA 02043 (617) 749-5100.

Structural Bearing Product Information

The Wabo-Fyfe Structural Bearing from Watson Bowman Acme is a low-profile, high-load bearing that uses Bonafy elastomeric rotational elements to accommodate the load and rotation of a structure. A centrally located shear restriction mechanism allows structural rotation but prevents shear from being applied to the elastomer. Horizontal structural movements are handled by the sliding of virgin PTFE against specially polished stainless steel. The bearings are manufactured in three basic configurations, depending on the movement required for the *particular application:* A fixed bearing is used where there is rotation but no horizontal movement; a unidirectional bearing restricts movements to one direction; and a multidirectional bearing accepts movements regardless of direction. Design compressive stress is approximately 5,000 psi plus/minus 5% and the design deflection is approximately 10% of element thickness.

For more information, contact: Watson Bowman Acme, 95 Pineview Dr., Amherst, NY 14120 (716) 691-9239.

Free Expansion Bearing Design Aids

A new manual for the design of structural fixed and expansion bearings for bridges and buildings has been released by Voss Engineering Co. The design manual, which features performance curves and design parameters for SORBREX and FIBERLAST based bearings, is the result of a recently completed test program. The high shear modulus, low deflection characteristics of these bearings offer the structural design engineer an alternative to other softer elastomeric bearing pad products. SORBTEX is made of layers of cotton polyester fabric impregnated with oil resistant synthetic rubber. The material has a minimum ultimate compressive strength of 10,000 psi, a shear modulus (G) of 450 psi at 33% pure shear, and a Shore A hardness of 90 durometer. FIBERLAST is a ROF pad material made of high-quality ozone-resistant virgin elastomer combined with synthetic fibers. This material has a minimum ultimate compressive strength of 8,000 psi, a shear/slip modulus (G) of 230 psi, and a shore Shore A hardness of 80 plus/minus 5 durometer.

The company's structural expansion bearing lower elements are made from laminations of SORBTEX or FIBERLAST bonded to 1¼"-thick steel plates recessed to contain a sheet of TFE. The upper elements are made to customer specifications. A new company manual provides information on bearings with thicknesses greater than 1", and is a companion to a 1989 design manual that covered bearings with thicknesses of ¼" to 1". The manual also includes design information about NEOSORB and Voss Slide Bearings.

For copies of these manuals and a free software design disk, contact: Rick Voss, Voss Engineering Co., 6965 North Hamlin Ave., Lincolnwood, IL 60645 (708) 673-8900.

AISC Publications

Engineers can reduce the time required for the analysis of two-, three-, and four-span continuous highway beam bridges with AISC's *Moments, Shears, and Reactions for Continuous Highway Bridges* design aid (88 pps.–$16.00). Tables of maximum moments, shears and reactions are presented for 456 continuous highway bridge spans, encompassing the full range of beam and usual plate girder bridges. The tabulated values and corresponding impact coefficients, based on one lane of AASHTO HS20-44 live loading, conform to the provisions of the 1983 AASHTO specifications. For problems involving special loading, corresponding tables of influence coefficients are presented.

To order a copy, send $16.00 + $4.00 postage and handling, to: AISC, P.O. Box 806276, Chicago, IL 60680-4120, and specify publication T106, or call (312) 670-2400 ext. 433.

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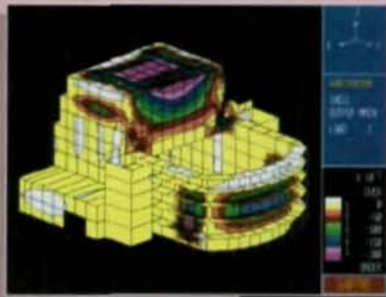
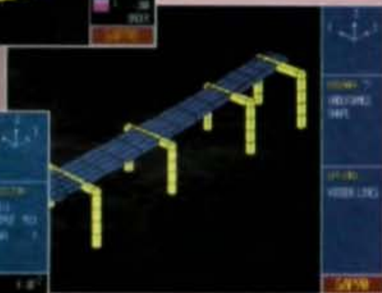

Dynamic Analysis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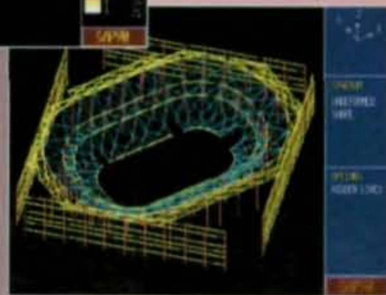
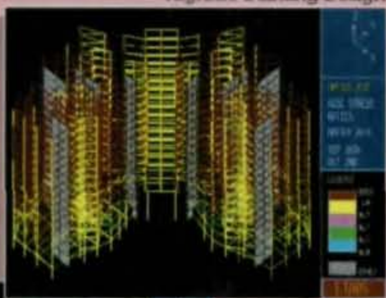
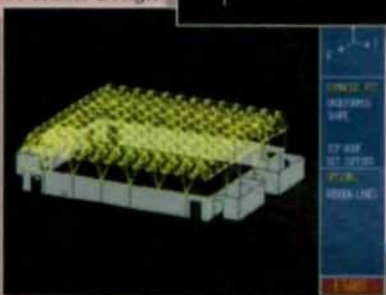

Complex Frame Analysis

Highrise Building Design



Space Frame Design



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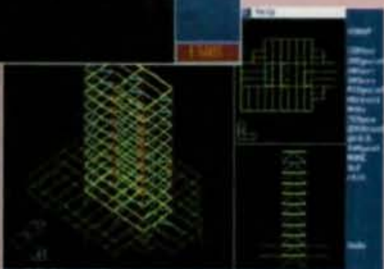
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