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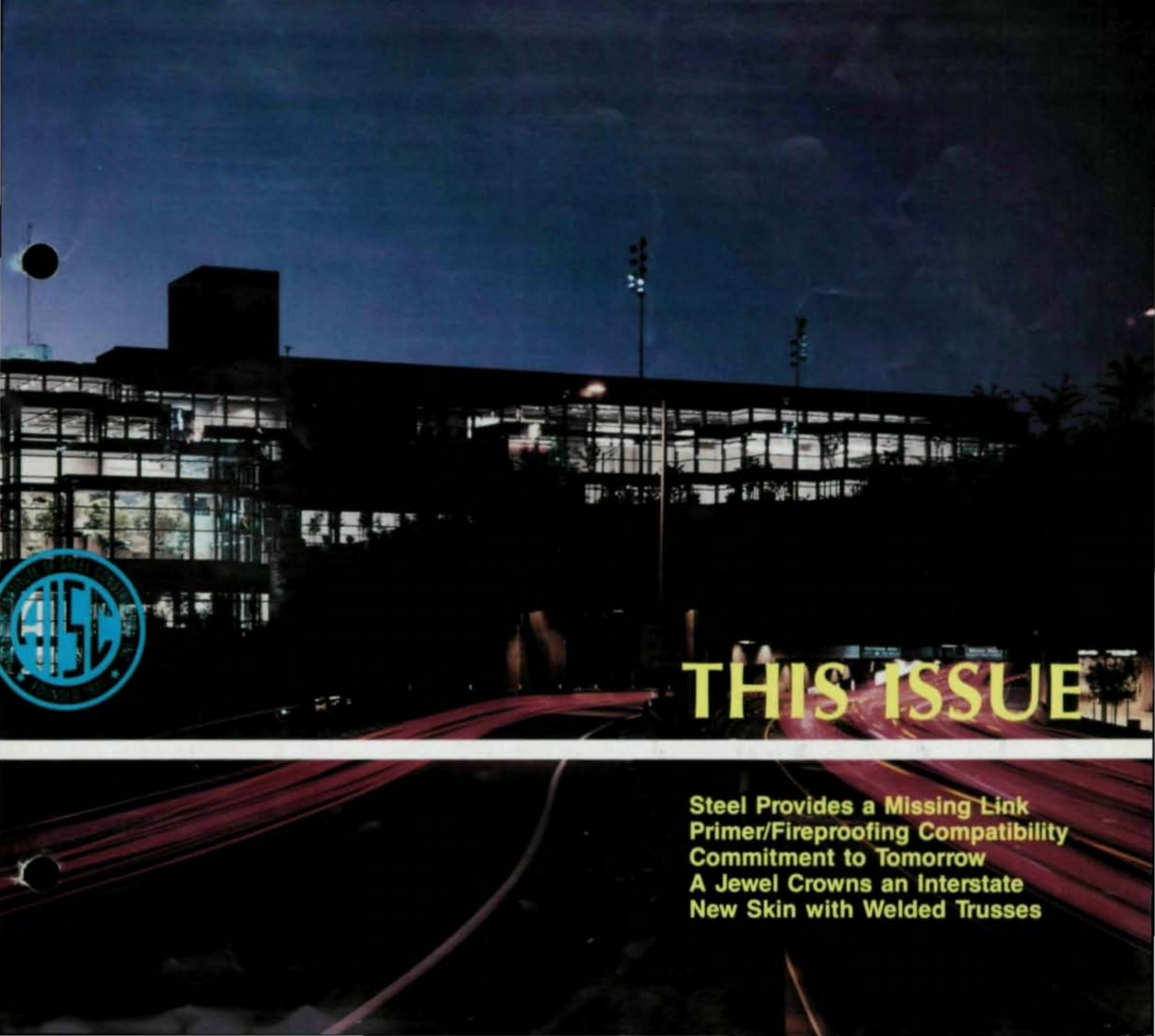
MODERN STEEL CONSTRUCTION

NUMBER 4 • 1989



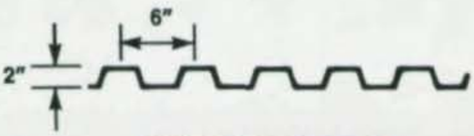
THIS ISSUE

Steel Provides a Missing Link
Primer/Fireproofing Compatibility
Commitment to Tomorrow
A Jewel Crowns an Interstate
New Skin with Welded Trusses

UNITED STEEL DECK, INC.

DECK DESIGN DATA SHEET

No. 12
FORM DECKS



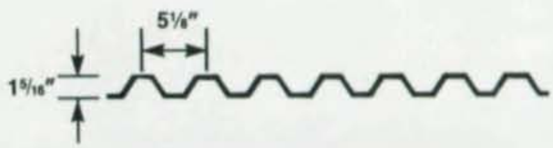
SECTION PROPERTIES

GAGE	METAL t	I	Sp	Sn
24	.0239	0.231	0.192	0.206
22	.0295	0.299	0.253	0.270
20	.0358	0.378	0.326	0.346
18	.0474	0.527	0.471	0.497

2" x 6" 30" COVER
EXTRA HEAVY DUTY FORM

UF2X

- Ribs can accept a ¾" stud
- Available vented (UF2XV) for lightweight fill roofs



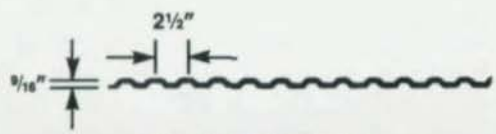
SECTION PROPERTIES

GAGE	METAL t	I	Sp	Sn
26	.0179	.071	.086	.096
24	.0239	.101	.125	.141
22	.0295	.130	.164	.183
20	.0358	.164	.212	.225

1$\frac{5}{16}$" x 5$\frac{1}{8}$" 36" COVER
HEAVY DUTY FORM

UFX-36

- Ribs can accept a ¾" stud
- Available vented (UFXV-36) for lightweight fill roofs



SECTION PROPERTIES

GAGE	METAL t	I	Sp	Sn
28	.0149	.012	.036	.039
26	.0179	.015	.046	.048
24	.0239	.022	.070	.070

$\frac{9}{16}$" x 2$\frac{1}{2}$" 30" COVER
STANDARD FORM

UFS

- Diaphragm values are available for all three form decks.
- Composite beam and girder tables are available for UF2X & UFX-36.
- A complete line of bridge form decking is also produced by USD.

MODERN STEEL CONSTRUCTION

VOLUME XXIX • NUMBER 4 1989
JULY-AUGUST 1989

American Institute of Steel Construction, Inc.

The Wrigley Building
400 North Michigan Avenue
Chicago, Illinois 60611-4185
Phone: 312 / 670-2400

On our cover
Washington State Convention and Trade Center, Seattle, Story on p. 28

Steel Notes

NEW LECTURE SERIES: ALLOWABLE STRESS DESIGN IN STEEL

Commencing in September 1989, and continuing through early 1990, AISC is featuring a six-lecture program that reviews the Allowable Stress Design method. Over 60 cities will be covered by this seminar which will particularly emphasize the new Ninth Edition *Manual of Steel Construction* as well as the newly adopted 1989 Specification for structural steel. For more information contact AISC 312/670-5422.

T. R. HIGGINS LECTURESHIPS

The 1990 T. R. Higgins Lectureship Award competition deadline is November 10, 1989. This program, in its 19th year, recognizes lecturers and authors whose published technical papers are considered an outstanding contribution to structural steel literature.

The competition prize is a $5,000 cash award and a framed certificate. For more information contact AISC at 312/670-5406.

WINNERS OF 1989 SCHOLARSHIPS

Five scholarship winners were announced by AISC's Committee on Education. They are:

Stupp Bros. Bridge & Iron Scholarship—
Robert W. Miller
Ohio Northern University

AISC/USS Scholarship—
Alan Price
CCNY

AISC/Carolina Steel Scholarship—
Kevin L. Burton
The Citadel

AISC/Fred R. Havens Scholarship—
Teresa Pfortmiller
Univ. of Kansas

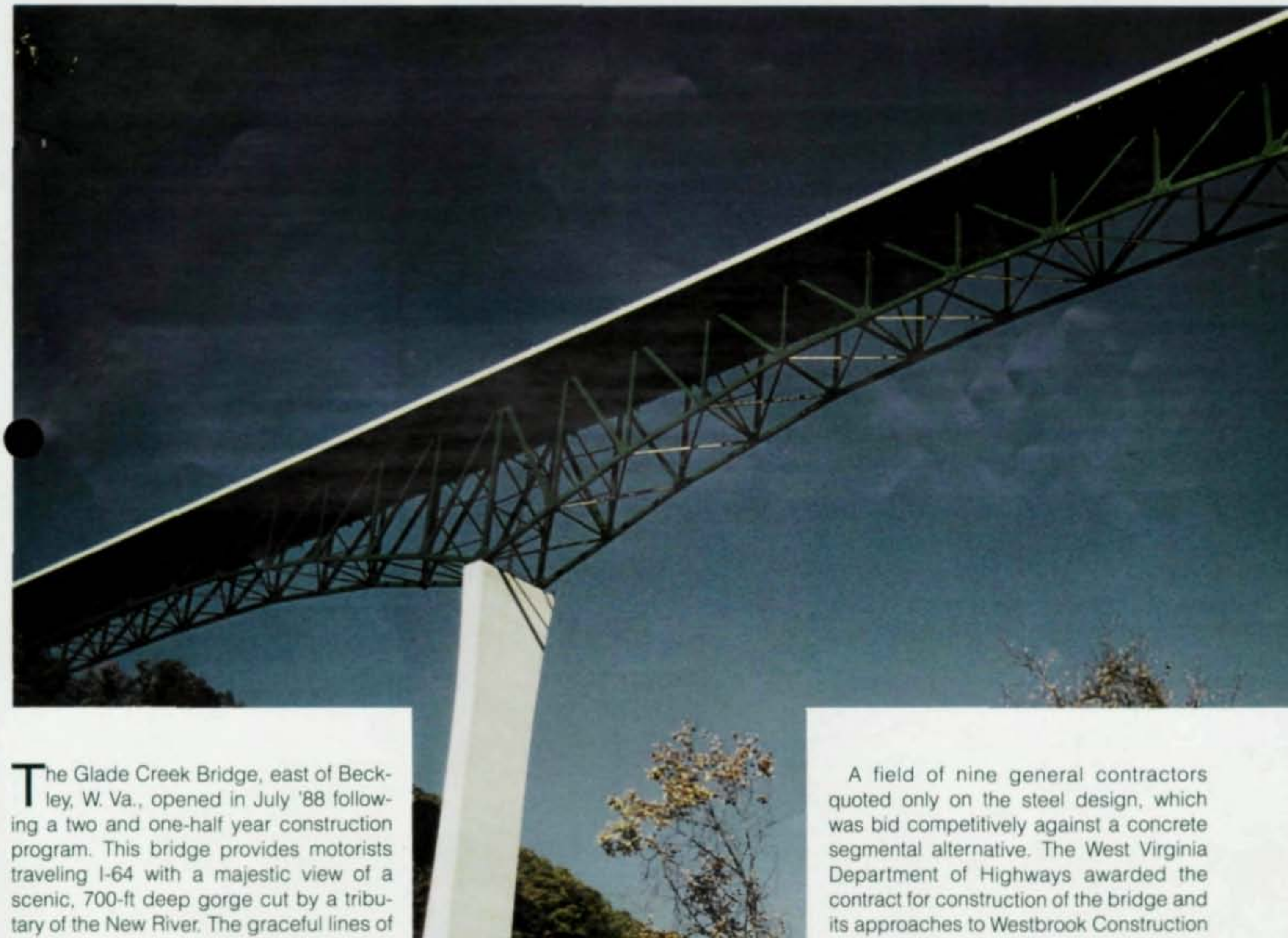
All recipients receive a cash award to support their educational costs for their senior year of undergraduate work.

Innovative Engineering

GLADE CREEK BRIDGE

Steel Fills a "Missing Link"

by Thomas D. Jenkins

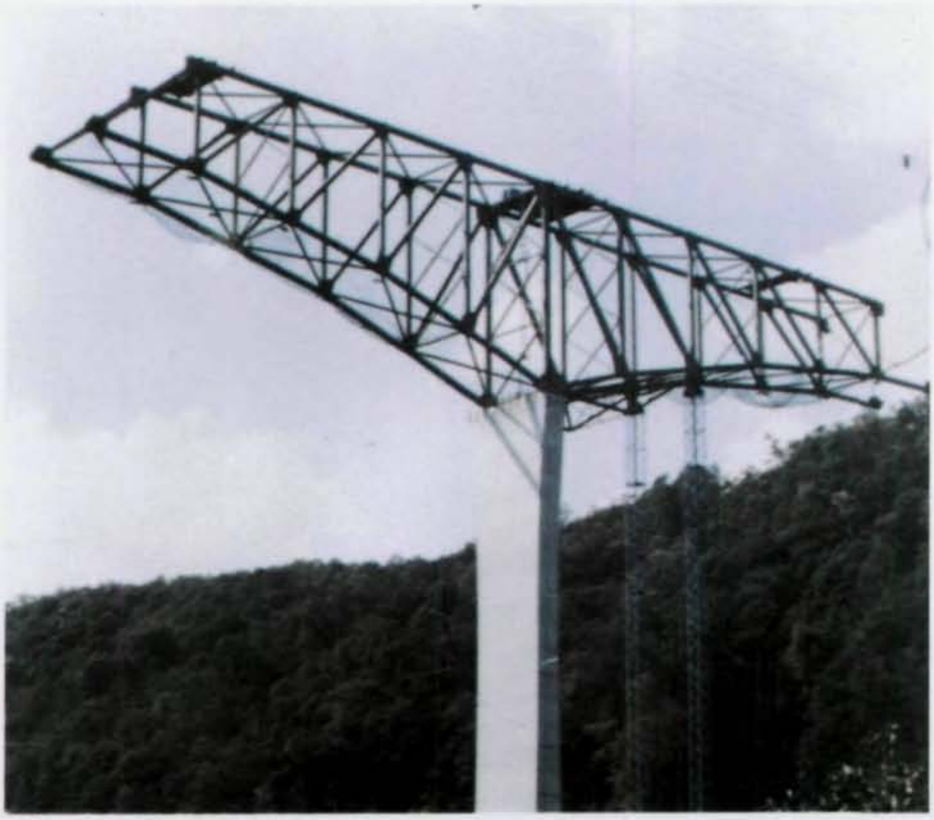


The Glade Creek Bridge, east of Beckley, W. Va., opened in July '88 following a two and one-half year construction program. This bridge provides motorists traveling I-64 with a majestic view of a scenic, 700-ft deep gorge cut by a tributary of the New River. The graceful lines of the bridge's arched trusses and monumental piers make the structure a notable landmark.

The bridge structural system is the product of the latest refinements in the design process for a form of steel truss. Engineers with the Baltimore-area office of Greiner, Inc., relied heavily on advanced computer analyses throughout the design process which employed new load factor specifications.

A field of nine general contractors quoted only on the steel design, which was bid competitively against a concrete segmental alternative. The West Virginia Department of Highways awarded the contract for construction of the bridge and its approaches to Westbrook Construction of Denver, Colo. The contractor tendered a bid of $29 million in late 1985. The bridge part of the total project was $25.5 million.

The bridge is a three-span, continuous-steel deck truss supporting two 36-ft roadways. The center span is 784 ft long flanked by 560-ft side spans. The superstructure extends beyond the ends of the main truss to stub abutments with steel multi-girder simple spans. Total length is 2,179 ft between abutment bearings.


Bridge erected in cantilever off its piers

The bridge features two reinforced concrete main piers which support the interior truss bearings. The main pier on the west slope is 351 ft high, with a 303-ft tall counterpart at the east slope. These piers are cast-in-place, two-cell box shafts whose form suggests an hour-glass shape when viewed from an oblique angle.

The truss ends are supported on 20-ft steel rocker bents carried on reinforced concrete pedestals. The entire bridge is founded on 3-ft dia. reinforced-concrete caissons drilled into rock.

Preliminary Studies

Greiner was selected to design the steel alternate for the crossing, with Sheladia Associates, Inc., of Riverdale, Md., as a minority subconsultant. Preliminary investigations established the layout of the truss and substructure. The principal concern was to achieve the most economical form for the truss and its substructure. A decision to avoid using steel with a specified yield strength greater than 50,000 psi was an important factor in economic comparison between a parallel chord arrangement and a more traditional haunched form.

Pier under construction

The studies showed the parallel chord truss would have required about 6% more steel and considerably larger compression chord members over the main piers with the material strength restriction. While the parallel chord layout offers repetition of detail and eliminates plane changes in the bottom lateral bracing, the curved lower chord for the haunched truss deepens the structure where the bending moments are largest and reduces the shear that must be carried by the web members. Furthermore, a haunched truss reduces the height of the main piers and lowers their bending moments from longitudinal forces. These factors collectively saved about $800,000 in estimated construction costs for the adopted design.

Structural System Details

The bridge deck for the eastbound and westbound roadways is an 8-in. thick, 77 ft-6 in. wide reinforced concrete slab. This slab is supported by nine lines of W30 × 99 rolled beam stringers. These stringers are composite with the slab and span the 56-ft truss panel spacing. The stringers are framed into transverse floor beams, with continuity provided by high-strength, bolted-flange splices. The top flange splice passes over the top of the floor beam while the lower flange is spliced using tees bolted through the floorbeam web.

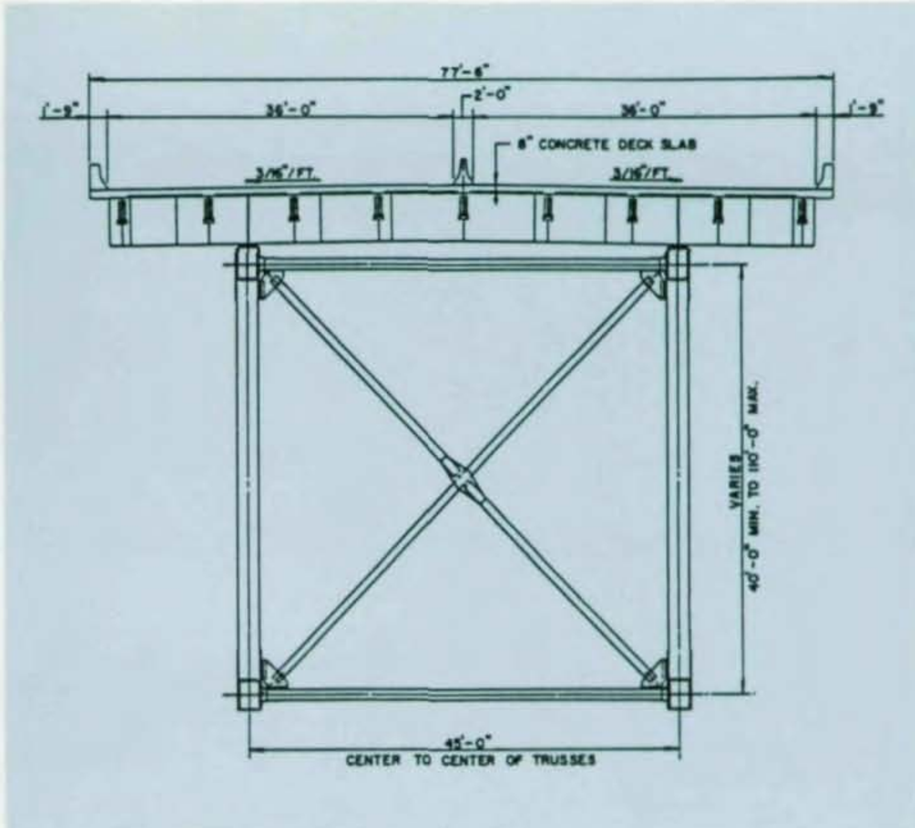
Dramatic steel span over deep gorge near Beckley, W. Va.

The floorbeams are welded plate girders with a 5-ft web depth. The web geometry parallels the cross-slope of the roadways which peaks at the bridge centerline. To simplify fabrication, stringer connections and intermediate stiffeners were framed normal to the floorbeam flanges and the floorbeam web depth was kept constant. A form web depth is advantageous because it permits simultaneous parallel cutting of the web's upper and lower edges and keeps all stiffeners the same length.

The floorbeams span 45 ft between truss centers and are supported on steel-reinforced, neoprene bearings placed on the upper truss chords. These bearings permit rotation in all directions and longitudinal movement of the floor system above the main trusses. Transverse floorbreaks divide the floor into seven units, typically five panels in length. Shelf-type, hinged joints near a floor beam provide for expansion of the stringers at these floorbreaks.

The main span continuous truss has a parabolic curve in the lower chord and Pratt-type framing for the web members. At 56-ft increments, the truss panels are relatively long. This panel length minimized the number of members and connections while keeping the member sizes at reasonable dimensions for fabrication, shipping and erection.

The upper and lower chords of trusses



Cross section of Glade Creek Bridge

are box-shaped members with two 37-in. main vertical plates varying in thickness from a minimum of 7/8 in. near a contraflexure point to 4 in. in the lower chord at the interior reactions. The main plates are continuously connected to top and bottom cover plates with fillet welds. The diagonals and taller vertical members are also box-shaped, with depths ranging from 22 to 44 in. The shorter verticals are H-shaped members with two main plates fillet-welded to a single web plate. All members widths are 28 in., the standard selected as the inside dimension between gussets at connections. The box-shaped members are closed by seal plates just inside the end connections. Initial and maintenance painting of the inside surfaces was thereby eliminated.

Members are connected at the truss joints through gusset plates using 1 1/8-in. dia. A325 high-strength bolts. Chord splices include inside splice plates, as well as top and bottom cover plate splices. The cover plate splices use smaller, 7/8-in. dia. bolts.

The upper and lower main-truss chords each have lateral bracing systems. Sway frames are provided between the vertical members at each support, adjacent to each support, and at alternate panels elsewhere. All bracing members are 14-in. wide-flange shapes in a Warren configuration. Bracing connection sizes were minimized by arranging convergence of the diagonal working lines on the face of the main truss rather than the chord center. While this positioning gives some minor out-of-plane bending to the main members, it virtually eliminates local plate bending from introduction of the bracing forces into the member, avoids extra internal stiffening and loads the connection angles with the truss faces primarily in shear.

The bridge required 7,000 tons of structural steel. The truss was designed with ASTM A572, Gr. 50, for virtually all main and bracing members and connections. Approximately 10% of the structural steel is ASTM A36. Field connections are made entirely with high-strength bolts. Contact surfaces of all bolted connections were shop-coated with inorganic zinc-rich paint to maximize slip resistance and minimize the number of bolts used.

Pier Designs

Steel rocker bents on concrete pedestals were selected to support the truss ends. These 20-ft links can accommodate the longitudinal movements of the truss with simple details and permitted convenient installation after closing adjustments completed the trusses.

Each 4,400-ton interior truss reaction is carried through a 20-in. dia., high-strength forged-steel pin to a base weldment mounted on the caps of the two main piers. The trusses are fixed to the tops of these two concrete main piers.

The outside shape of these two-cell shaft piers varies from top to bottom; however, the variation was arranged such that only one principal dimension would change within any lift during construction. This constraint was placed on the pier shape since it simplified the problem of form advancement and alignment. The constructibility of the design was confirmed when the 28 lifts for th 330-ft box section of the taller pier were all placed within ten weeks.

The upper 100 ft of the pier has a transverse parabolic flair arranged on four 25-ft chords. The 58 ft-2 in. × 15-ft pier top reduces to 44 ft × 15 ft through this region. The next 25-ft segment has constant outside dimensions, yet the inside transverse measurement continues to reduce, thereby giving a thickness transition to the sidewalls. Below the top 125 ft, all transverse dimensions were held and the longitudinal dimensions varied on a parabolic flair for four 24-ft chords. The inclinations of the walls was then maintained for the remainder of the pier height. The taller Pier No. 2 has five more 24-ft chords, flaring to a base dimension of 44 ft. Pier No. 3 is 48 ft shorter, with a base dimension of 44 ft × 26 ft.

The perimeter and internal web walls are 16 in. thick throughout the upper por-

tions of the shafts. The sidewalls increase to 21 in. in the lower pier. The internal web wall provides an efficient method of stiffening the long transverse walls when compared with increasing their thickness and using a single-cell arrangement.

The shaft walls are conventionally reinforced with heavier vertical reinforcing required in the shorter pier because its greater stiffness attracts more longitudinal force. The reinforcement is concentrated toward the outside corners to maximize the usefulness of the steel under the skew angle lateral loadings that controlled the pier design.

The shaft cap is solid below the truss bearing for a 7 ft-6 in. depth. This section is rather heavily reinforced to distribute the truss reactions into the pier walls. Special design investigations of the pier tops were made to assure adequate reinforcement for splitting forces. These analyses resulted in substantial transverse reinforcement for the cap itself and increased horizontal reinforcement in the walls for 6 ft below the cap.

The analysis of forces normal to the pier walls originating from their plane changes identified a need for additional horizontal reinforcement in bands centered on these kinks.

The specifications called for all pier concrete above the footings to have a 5,000-psi compressive strength vs. 3,000 psi specified for the footings. The footings for both main piers are 10 ft thick and each is founded on 38 3-ft dia. caissons drilled into rock and filled with 5,000-psi reinforced concrete.

Structural Design Analysis

The truss superstructure was designed in conformance with AASHTO's recently adopted *Guide Specifications for Strength Design of Truss Bridges*. The design was facilitated by using a truss analysis computer program developed by Greiner. The program performs a complete dead- and

Steel design on gigantic bridge was selected in competition with a concrete alternative.

live-load analysis and combines these data with user-supplied information on member loadings under wind, temperature, traction, etc. Group loads are then assembled and factored and the adequacy of each member evaluated in accordance with AASHTO specifications. The program also checks each member to estimate its critical wind speed for vortex-induced oscillations. Additional computer runs were made to identify all fracture-critical tension members.

The analysis for the substructure was based on a three-dimensional model of the whole bridge as a frame. An in-house, modified version of STRESS generally was used and supplemented for dynamic analysis with STRUDL software from McDonnell Douglas.

For the three-dimensional model, the truss was represented as a spine of nodal points with connecting members whose stiffness was selected so that the actual stiffness of the assemblage of members between two truss panels would be simulated closely. The piers were also idealized as spine-type members. This model was subjected to all static loadings relevant to the design of the piers. From these results, the loading cases governing the design were identified. The reduction of pier stiffness expected under these loadings, as a result of concrete cracking, was then determined. The stiffness of the pier spines was reduced accordingly and a second-order analysis performed under the most severe loading cases. This analysis showed the primary bending moments would be increased up to 10% because of the structure deflections. Consequently, all pier section, first-order design moments were increased by this amount.

Evaluation of the resistance capacity of the pier shaft at various elevations and with various reinforcement layouts was facilitated by another computer program developed by Greiner. This program performs an analysis of any shape reinforced concrete section by either ultimate strength or working stress methods.

The modified STRESS program was also used to perform a multimode spectral analysis of the bridge under an earthquake design loading of 0.1 g. This analysis relied on structure mode shapes and frequencies determined using STRUDL. The height of the main piers gives the structure long period fundamental modes which prevent the generation of large earthquake loads. The structure design was not governed by earthquake loadings. However, adequate accommodation of earthquake displacements was a factor in the selection of rocker bents for the truss end reactions and in the design of the bridge seats and deck joints at the abutments.

Closing the Final Gap in I-64

The bridge filled the last remaining gap along I-64. Westbrook began construction early in 1986 by building a network of access roads to the base of the piers, which included construction of a short, steel beam bridge across Glade Creek. All excavated material had to be hauled out of the gorge which is partly in an environmentally sensitive state park.

Drilling for the foundation caissons started after completing the excavations and sealing the upper rock layers by grouting. Once the 38 caissons were placed for the main pier, the 10-ft thick footings could be built to complete the foundations.

The two-cell shafts were built in 12-ft and 12 ft-6 in. lifts using jumping formwork. Twenty-four lifts were required for Pier No. 3, the first one constructed. Twenty-eight lifts were necessary for Pier No. 2.

The materials for the piers were hauled in to the base of the shafts and then hoisted to the required height using an internal, self-climbing tower crane temporarily erected within one of the cells. The walls of the cell supporting the crane had to be strengthened with additional bands of transverse reinforcement to support the horizontal loads imposed where the crane's tower was wedged in.

The caps were constructed on stay-in-place metal formwork carried on steel beams spanning the 12 ft-4 in. spacing between the transverse walls. The heavily reinforced 7 ft-6 in. caps were placed in two lifts.

Harris Structural Steel, of South Plainfield, N.J., fabricated the steel superstructure. These components were then hauled to a staging area at the bridge's west approach. American Bridge served as the erector and used an aerial cableway to span the gorge. In addition to erecting the superstructure, the cableway served to remove the tower crane from the top of the main piers once the shafts were completed.

The superstructure was erected in balanced cantilever, starting off the main pier and a tall falsework tower two panels away on the up-slope side. The falsework towers were tied into the pier tops by a hog truss on which the initial two chord members were placed. The temporary towers carried only vertical loads and all lateral and torsional reactions had to be completely resisted by the concrete shafts.

Balanced construction proceeded until the superstructure landed on a second falsework tower located two panels away from the truss end. After landing on this support, the first tower was removed and re-erected to build the second half of the bridge.

Once both truss halves were completed, the span was closed. This was accomplished by first lowering the elevation of the end falsework to produce equal gaps in the upper and lower chords and to align the center of the main span. Jacks mounted on the lower chords then pulled the two halves together by deflecting the tall piers. The upper and lower chords were then bolted. The falsework was raised to install the end rocker bents before setting the bridge on its permanent bearings and removing the falsework.

With the truss completed, the floor system was erected, progressing from the west end. The concrete deck followed using normal cast-in-place procedures for steel, multi-beam bridges, with placement also advancing from the west.

The steel bridge forged the final element in West Virginia's Interstate Highway System. Blending beauty with functionality this dramatic span filled the missing link in an important route connecting Norfolk, Va. with St. Louis, Mo. □

Designer/Structural Engineer
Greiner, Inc.
Timonium, Maryland

General Contractor
PCL Civil Constructors, Inc.
Plantation, Florida

Steel Fabricator
Harris Structural Steel Company
South Plainfield, New Jersey

Steel Erector
American Bridge
Pittsburgh, Pennsylvania

Owner/Construction Manager
West Virginia Department of Highways
Charleston, West Virginia

Thomas D. Jenkins, P.E., is vice president of Greiner, Inc., Timonium, Maryland

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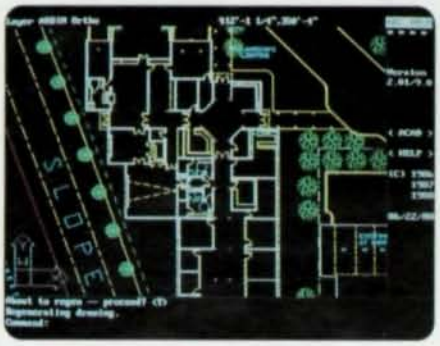

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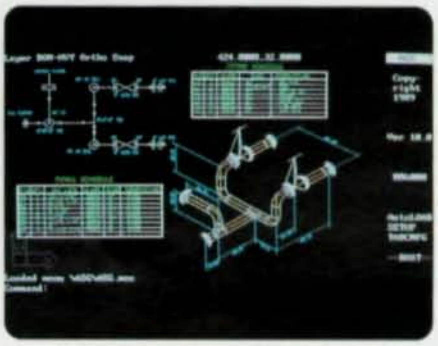

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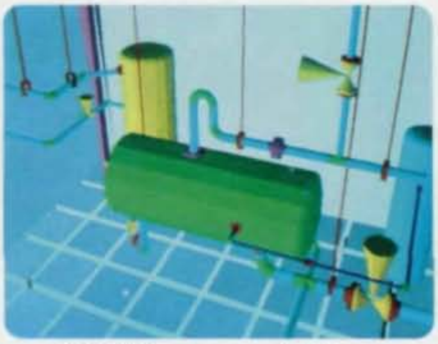

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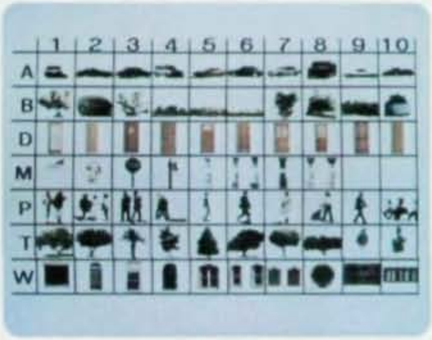

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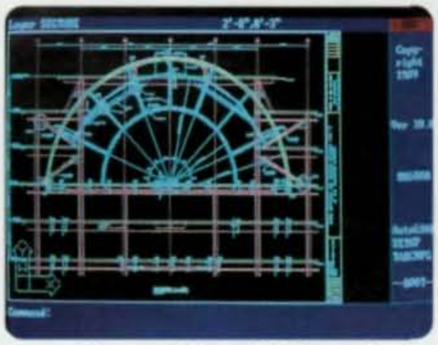

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PRIMER/FIREPROOFING COMPATIBILITY

Underwriters Laboratories, Inc. recently issued new guidelines on the application of spray-applied, fire-resistive coatings to painted structural steel framing. These new guidelines can increase costs significantly when these coatings are applied to structural steel framing that has been painted. As a result, any projects that may involve the application of spray-applied, fire-resistive coatings to painted structural steel should be assessed carefully in light of the new UL guidelines.

Background

For many years, the American Institute of Steel Construction specifications have not required the use of paint when structural steel framing "will be concealed by interior building finish or will be in contact with concrete." Statements to this effect are included in the current editions of both the ASD (Sect. 1.24.1) and LRFD (Sect. M3.1) specifications published by AISC. The Commentary to the LRFD specification supports this provision with the following discussion:

> "The surface condition of steel framing disclosed by the demolition of long-standing buildings has been found to be unchanged from the time of its erection, except at isolated spots where leakage may have occurred. Even in the presence of leakage, the shop coat is of minor influence."

In spite of the AISC specifications, some paint manufacturers are promoting the use of primers beneath fire-resistive coatings. In response to an apparent increase in the use of primers in some geographic regions, Underwriters Laboratories has conducted a limited study of the adhesion of spray-applied, fire-resistive coatings to painted structural steel. This study has resulted in the issuance of new guidelines in the *1989 UL Fire Resistance Directory* (Design Information Section—Sprayed Material).

UL Guidelines

The new UL guidelines specifically permit the application of spray-applied, fire-resistive coatings to painted steel with these significant restrictions:

- For beam applications, the web depth cannot exceed 16 in. and the flange width cannot exceed 12 in.
- For column applications, neither the web depth nor the flange width can exceed 16 in.

Within these size limitations, fire-resistive coatings may be spray-applied to painted structural members, provided that bond tests indicate the fire-resistive coating will adhere adequately to the painted steelwork. Depending on the characteristics of the paint and fire-resistive coating, a bonding agent may be required. If these size restrictions are exceeded *or* the fire-resistive coatings fails the required bond tests, the structural shape must be wrapped with steel lath. In some cases, deck discs can be welded to the steel framing in lieu of steel lath. Either alternative will result in cost increases that should be considered prior to specifying the use of shop or field primers. It should also be pointed out that UL is considering the establishment of a testing, classification and follow-up inspection service for primers that can be applied to structural steel prior to the application of fire-resistive coatings. As a result, specific primers may be available that can be used beyond the previously described limitations.

So as to indicate the potential impact of the new UL guidelines, the following tabulation indicates some of the wide-flange shapes that *will* meet the specified size restrictions:

	Beams	Columns
6" Series:	All	All
8" Series:	All	All
10" Series:	All	All
12" Series:	W12×65 & lighter	All
14" Series:	W14×82 & lighter	W14×257 & lighter
16" Series:	All	—
18"-40" Series:	None	—

Note: This tabulation was developed based on the assumption that web depth is equal to selection depth less twice the flange thickness.

As can be seen, many structural shapes will *not* meet the UL size restrictions and either steel lath or deck discs will be required if the steel is painted with a primer which is not UL-classified for use beneath the specific spray-applied, fire-resistive coating.

Summary

- AISC does not require the use of primers for structural steel that will be concealed by interior building finish or in contact with concrete.
- UL does not require the use of primers beneath spray-applied, fire-resistive coatings, except for a limited number of very specific designs. In general, UL specifies the steel must be clean and free of dirt, loose scale and oil.
- The *1989 UL Fire Resistance Directory* should be reviewed carefully before a primer is specified in conjunction with a spray-applied, fire-resistive coating.
- If a primer is to be used on structural steel that will be protected with a spray-applied, fire-resistive coating, both the paint manufacturer *and* the manufacturer of the fire-resistive coating should be contacted for guidance. Depending on the circumstances, bond tests and/or mechanical anchorage will be required.
- It should be emphasized, this issue only affects structural steel framing. It does not impact the use of pre-painted or galvanized steel decks specifically recognized in UL designs, nor does it affect the use of painted steel joists that are specifically included in UL designs. □

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POLYMER SCIENCE BUILDING
Commitment to Tomorrow

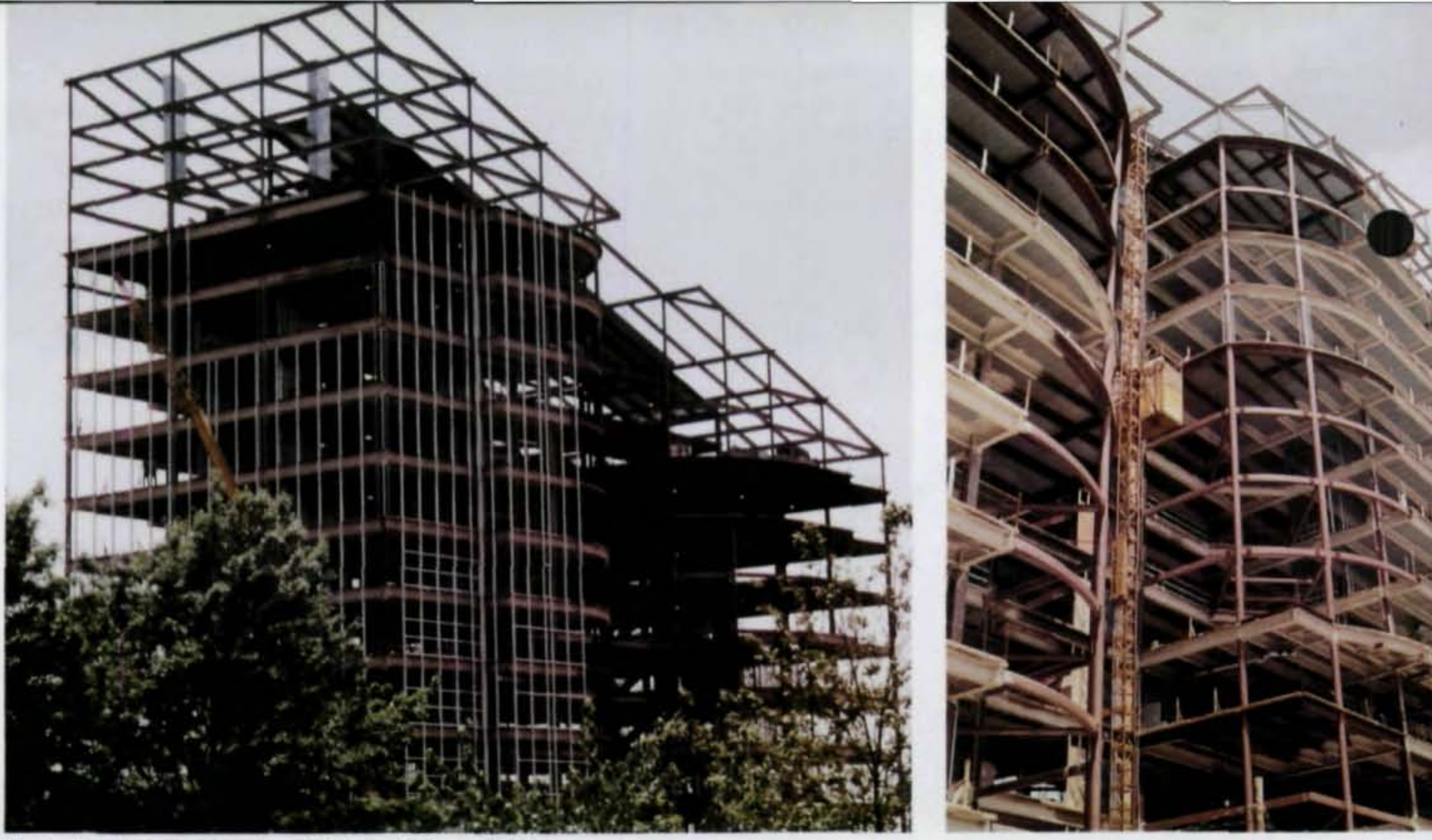
by Shakti P. Gautam

The Polymer Science Building, located on the campus of the University of Akron, was designed to be the best polymer research facility and home of the best polymer research program in the world. The commitment for a new facility was very logical for the university which, at the time, had one of only three such programs in the U.S., and whose neighboring corporations were investing heavily in polymer research.

The university's goals were to construct a state-of-the-art facility, conducive to research and education, while making a dynamic architectural account to the community about its commitment to tomorrow's technologies. It was also important that the users of the facility be safeguarded from the hazardous effects of toxic chemicals, pressurized gases and potential laboratory explosions. With these guidelines, it was imperative to assemble a design team able to work together to define, and continuously refine, the relationship of architectural form and mechanical requirements to structure, while ultimately designing a unique and functional facility.

The final design of the facility consists of two towers, nine and 12 stories, framed in steel and wrapped with a glass curtain wall. Both towers, containing laboratories and offices, are linked together at each level by a stair tower and commons area. A steel-framed canopy rises 60 ft above the 12-story tower and slopes down to the nine-story tower, integrating the two into a

Model of Polymer Science Building



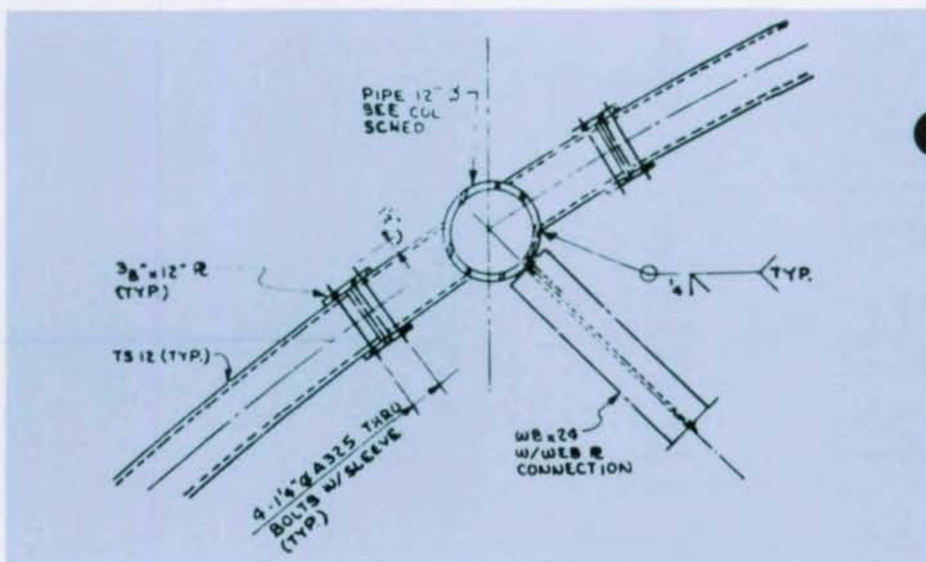
Views of intricate steel framing of Polymer Science Building. Below, typical atrium connection

single building with an oblique roof. Adjacent to the east wall of the 12-story tower is a three-story auditorium.

Steel a Design Choice

In preliminary design stages, the use of plastic (polymer) members as structural elements was considered, but, because of required design criteria, this was technically and economically unfeasible. More conventional materials were then considered and steel chosen because of its cost and flexibility.

Various structural systems were developed and evaluated during preliminary stages. Initial studies focused on using the vertical circulation cores of each tower to provide the required stiffness. But it was determined the core walls contained too many openings to be used efficiently. Moment-resisting frames, exterior braced bays and combinations thereof were then evaluated, with the decision that moment-resisting frames provided the most economical and least obtrusive system. And, by featuring moment-resisting frames in both directions, it was possible to utilize the stiffness of both towers to overcome the slenderness of each tower. Furthermore, by locating the frames primarily on interior column lines, the depth of the spandrel beams, as seen through the curtainwall, was kept shallow to emphasize the visual lightness of the structure.



The project constraints dictated a shallow floor construction depth which required careful selection of member sizes. Steel framing permitted a shallow and uniform depth of construction in the laboratory areas, with an extensive network of mechanical ducts and hoods located within the ceiling. Deeper girders along the rigid frames permitted supply and return ducts to penetrate the beam webs.

Unusual Structural Design

Some of the architectural features on the project that required unique structural design and detail applications include the atrium areas and the main atrium staircase on the lower levels. The open atriums at the inside corners in both towers are three to five stories in height and alternate between various bays from ground level to roof. Because of the aesthetic requirements, pipe columns and tubular spandrel beams emphasize the lightness of the framing. The 12-in. diameter pipe columns were detailed in lifts of three to five stories to avoid expensive field-welded column splices and permit quicker erection. Curved structural tubes were used for spandrel beams in the atriums because they provided the most economical torsional section properties required to carry

Photo of model of new science building

the curved glass curtainwall. And, by employing tubes, it was possible to detail shop-fabricated stubs off the pipe columns for quick erection with simple, field-bolted splices of the spandrels.

The main atrium staircase rising from the entry level to the second and third floors is located in the middle of a five-story atrium. The unique feature of this stair is that intermediate landings between floors appear to be floating in the air approximately 15 ft from the face of support. The structural framing for the stair consists of structural tubes cantilevering off each floor to support the stair treads and landings in a scissors fashion. Tube members were again selected for their efficient torsional properties and pleasing appearance.

The Polymer Science Building, scheduled for completion next Spring, has about 150,000 sq. ft of floor area and a construction budget of $13.5 million, not including lab equipment and furniture. Not only did the cost of the 1,200 tons structural steel used come in below project estimates, but also it was erected two months ahead of schedule. □

Architects (joint venture)
Richard Fleischman Architects, Inc.
Lawrence Dykes Bower and Clancy Architects Planners
Harris/Day Architects, Incorporated
Cleveland, Ohio

Structural Engineer
Gensert Bretnall Associates
Cleveland, Ohio

General Contractor
Kokosing Construction, Inc.
Fredricktown, Ohio

Owner
State of Ohio Department of Administration Services and The University of Akron

Shakti P. Gautam is a project engineer with Gensert Bretnall Associates, Cleveland, Ohio.

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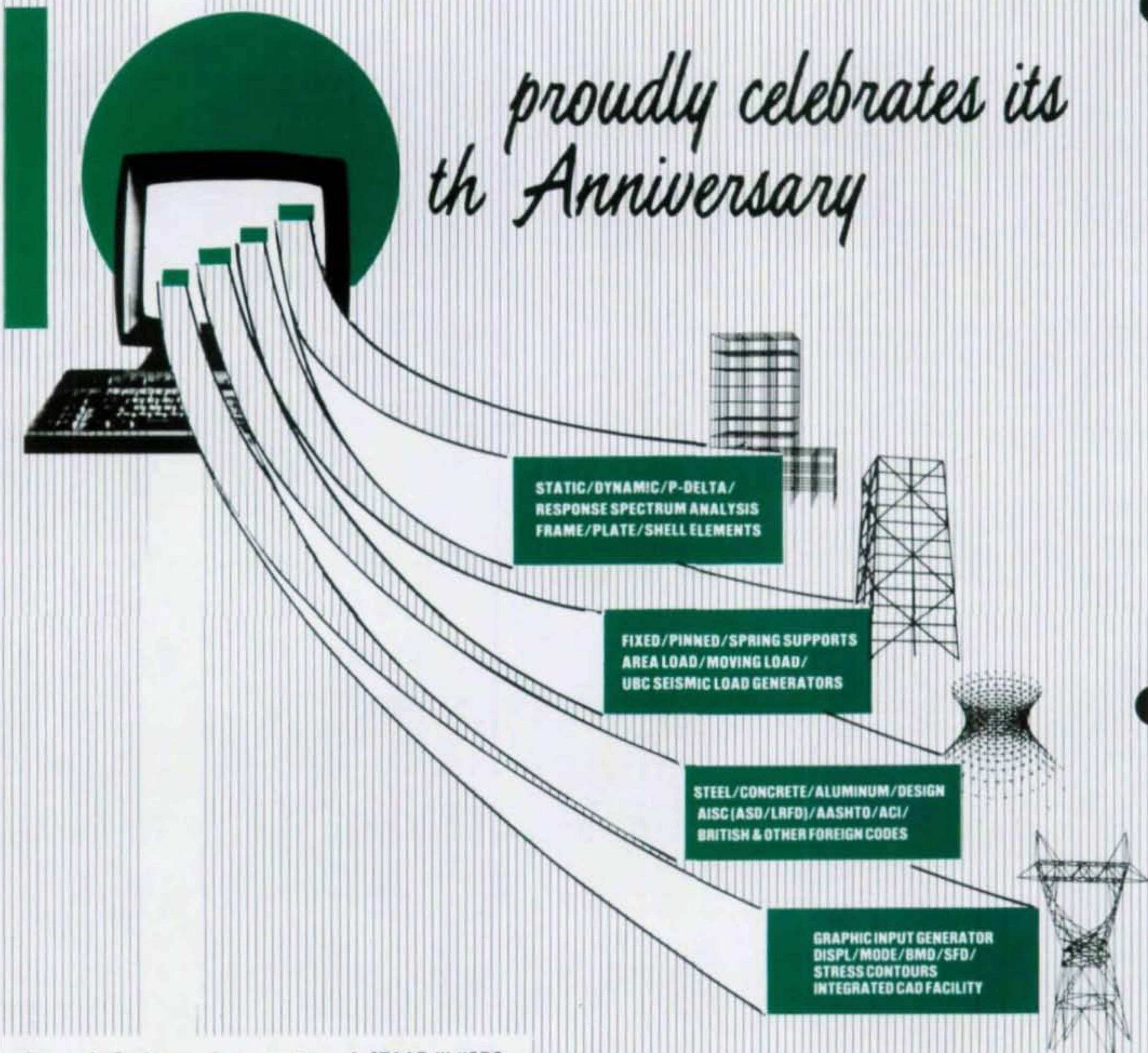

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WASHINGTON STATE CONVENTION CENTER

A Jewel Crowns an Interstate

by John B. Skilling, Jon D. Magnusson and Ad A. Gouwerok

Dramatic cascading lobby of Washington State Convention and Trade Center, Seattle.

The $130-million Washington State Convention and Trade Center—a 10-level, 1,000,000-sq. ft facility constructed over Interstate 5 in downtown Seattle—spans three city streets and 12 freeway lanes.

The structure is framed in structural steel. Sixteen major trusses up to 90 ft in depth carry the gravity loads, as well as wind and earthquake loads. The arrangement of these trusses permitted the development of three levels of parking for 1,000 cars below the exhibition floor.

In 1981, the Washington state legislature passed a bill for issuance of a bond to provide funding for the design and construction of the Washington State Convention and Trade Center. A design committee was chaired by James Ellis, a driving force behind many public projects in Seattle. At that time, a design team was chosen which consisted of TRA, the managing architectural firm on this most challenging project, and HNTB. The structural engineering was to be provided by Skilling Ward Magnusson Barkshire Inc. The first task this team had to face was site selection, narrowed down to three sites:

1. At Seattle Center.
2. Near the present stadium facility (the Kingdome).
3. Air rights over the I-5 freeway.

After trial designs and long and care[ful] economic study, the I-5 freeway site was chosen.

Freeway Site

Structurally, this was certainly the most challenging site. Only about 25% of the footprint of the Convention Center would touch down on available land; The remainder would span the busiest stretch of freeway in the Northwest, namely, 12 lanes of I-5. Several schemes were studied at that time, one of which was a concrete "lid" covering the freeway completely, similar to the nearby Freeway Park—only far stronger because of the far heavier loads needing to be supported. The convention center would be placed on top of this concrete lid. But this concept was rejected, because it could not be built within the budget.

Structural System

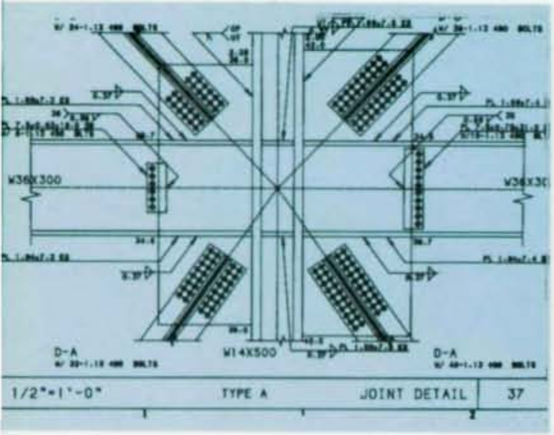
At that time, the State Convention Center joined forces with a public developer. Included in the project would be a retail area and a parking garage for about 800 cars. With this information available, the design team went to work. The parking garage gave an excellent opportunity to provide a most economical steel-truss base for the heavy convention center. It would be the parking garage which varied from one to three levels. Rather than supporting each level on its own trusses, the total depth of the parking levels was used. The trusses were to be one to three levels deep. This would create up to 45-ft deep trusses spaced 60 ft o.c. The 60-ft space between the trusses is to be used for an efficient parking layout. The bottom chord of the trusses would be in the lowest parking level and the top chord in the roof over the topmost parking level. Where the exhibition hall walls allowed, some trusses would actually be 100 ft deep, the top chord being in the roof of the convention center. The trusses would also act simultaneously as braced frames.

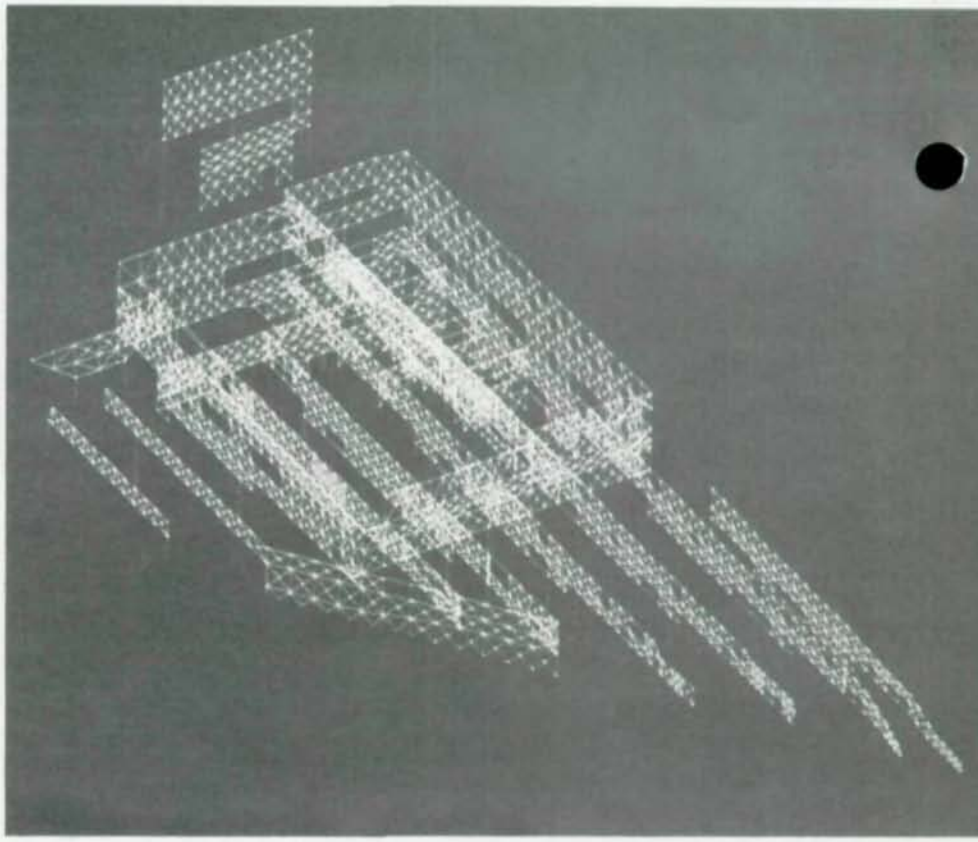
This top floor over the topmost parking level is also the convention center exhibition floor, able to carry loads of up to 350 psf or the AASHTO HS20-44 truck loading. Over this exhibition floor would be another floor of meeting rooms and a smaller exhibition hall able to carry loads of 150 psf. in addition to these heavy loads, the roof over the whole structure has been designed to carry a future park with a soil depth of three to four ft, enclosing an expanse of water as a feature of the park.

Economy and Efficiency

All this loading would have been economi-

Traffic moves at regular pace despite massive construction overhead.





Computerized design of connections
Over 19,500 tons of structural steel straddle freeway.

cally impossible to support were it not for the three-level deep steel trusses integrated in the parking structure below and the full structure depth 100-ft high trusses. Even though the structural steel supports these enormous loads and soars over a freeway with spans of up to 160 ft, the total weight of steel used on this project is 19,500 tons, which translates to about 35 psf of steel. An efficient parking system was accomplished by eliminating certain bracing panels, thereby creating strategically located driveways through the trusses from one aisle of parking to the next.

Seismic Design

An extensive seismicity study has been performed by Shannon and Wilson, geotechnical consultants on the Washington State Convention and Trade Center. It was decided the seismic design for the structure would be evaluated using two earthquake levels, similar to the Design Basis Earthquake (DBE) and Operating Basis Earthquake (OBE) recommended by the U.S. GSA (1978). The DBE is defined as an event having a recurrence interval of 200 years. It is expected that, although the structure would receive some damage in this earthquake, causing some members to exceed the AISC allowable stresses but not the steel yield stresses, collapse would not occur. The OBE is intended to represent an earthquake which is quite likely to occur during the life of the structure.

It is defined as having a recurrence interval of 50 years. Analysis for the OBE is intended to reflect the elastic behavior of the building with no structural damage. The stresses would not exceed the AISC allowable stresses. Given the above criteria, the geotechnical consultant determined the two spectra for the "design earthquake" and the "maximum credible earthquake." A dynamic analysis was performed on the structure using these two spectra.

A computer model was generated to study the structural behavior under both static and dynamic loadings. This model, which required 22,000 lines of input, was developed on our IBM 370 mainframe, using a version of STRUDL. The number of nodes and members in the model are 1,700 and 4,800 respectively. The STRUDL program calculated the member forces and a post-processor designed the members based on these forces. The member sizes were put back in the STRUDL model and reanalyzed. Redistribution of the stiffness redistributed the forces. An iterative process (four iterations) was used to come to the final sizes. The structure was analyzed for six static and 12 dynamic loading cases.

Redundancy and Ductility

To provide the safest possible structure, a very redundant system of cross-diagonal bracing was chosen. Wherever possible, the trusses and bracing elevation panels received cross bracing rather than a single brace, providing a high degree of redundancy. Further safety has been built into the structure by having all beam and column intersections in all trusses and braced frames moment-connected to each other, thereby providing a high degree of ductility in the system. Even though this moment frame was not required by code, it was decided to use this design philosophy to increase ductility and axial strut capacity.

As pointed out earlier, only 25% of the footprint of the structure touches the ground in a triangular area immediately to the northwest of the freeway. This area was used structurally as an anchor for the building. The anchor is a grid of 12-ft by 10-ft grade beams supported by 10-ft dia. drilled piers extending up to 100 ft into the ground. Diagonally across from this anchor on the other side of the freeway, another grade beam supported on 10-ft dia. drilled piers provides rotational stability, and at the same time allows for possible temperature movement of the structure toward and away from the anchor.

Architecturally Exposed Structural Steel

As with most convention centers or similar buildings, a dramatic expression of structure was sought. The lobby at the entrance into the exhibition hall is covered by a cascading roof, spanning up to 120 ft. The lobby area needed to be free of columns. The solution was a two-way arching moment space frame with all horizontal members 27-in. wide flanges and all vertical members 12-in. × 12-in. tube columns. All this is architecturally exposed structural steel.

Connection Design Economy

In recent years, concerns have been raised by the structural engineering profession, as well as different public agencies, regarding the fact the fabricator—and not the engineer of record—is designing connections.

Our firm has already been designing the connections on numerous major projects. This does not mean an increase in overall cost to the owner—actually, quite the opposite is true. When all the connections are completely designed and on the drawings, the fabricator/erector will not have any unknowns.

Another point to consider is that design parameters for connections are becoming more and more complex. They depend on a multitude of different loading combinations. So as to make economical connections, all

Truss erection roars on during night.

Top view of arch—exposed lobby steel.

the different design parameters should be taken into consideration. Because of their complexity, it is impossible to put all the criteria and loading combinations on the drawings in such a way as to be useful to the fabricator.

Convention Center Jewel

Presently, the convention center is open for business—on or close to the original schedule—with its first convention held in June 1988. The Washington State Convention and Trade Center is an economical asset to both the Seattle and the State of Washington, one of many jewels in the Emerald City.

Architect
TRA and HNTB

Structural Engineer
Skilling Ward Magnusson Barkshire Inc.

General Contractor
Paschen Contractors

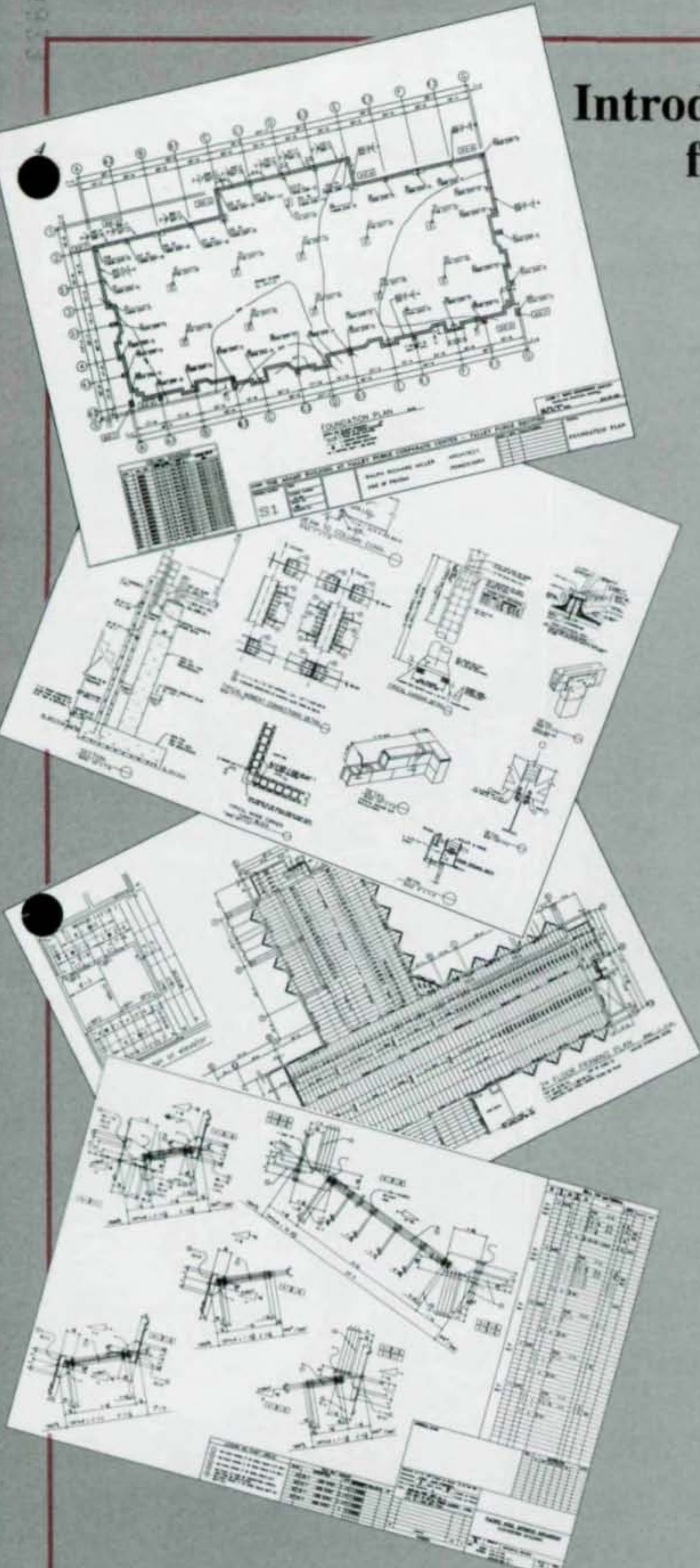
Geotechnical Consultant
Shannon and Wilson

Owner
Washington State Convention and Trade Center

John B. Skilling is chairman, Skilling Ward Magnusson Barkshire Inc., Seattle, Washington.

Jon D. Magnusson is chief executive officer of Skilling Ward Magnusson Barkshire Inc.

Ad A. Gouwerok is senior vice president, Skilling Ward Magnusson Barkshire Inc.



New Skin Profile Made Possible with Welded Trusses

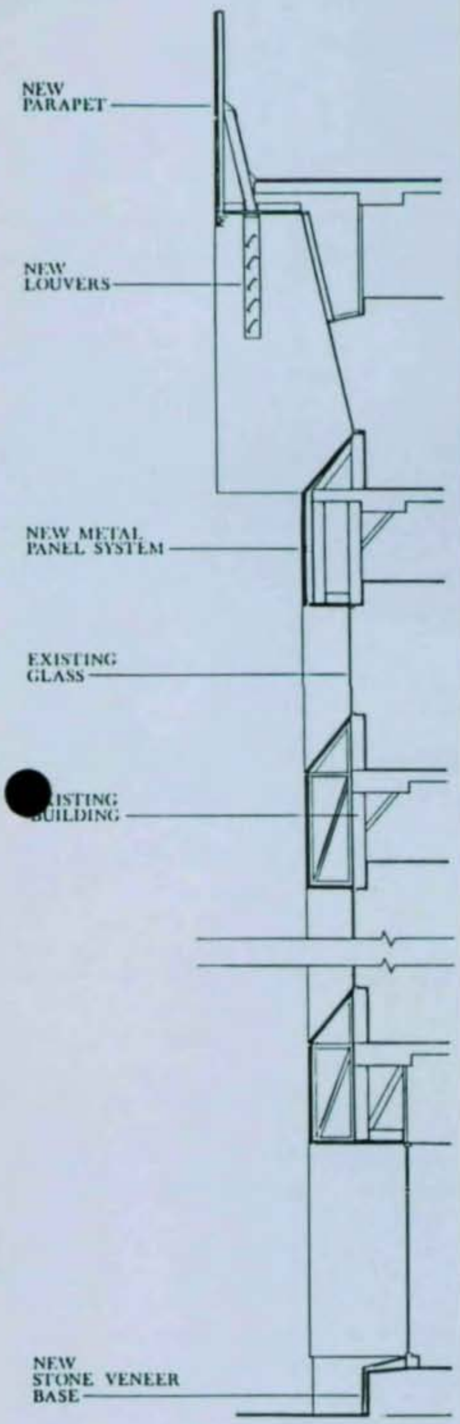
by K. S. Rajagopalan and Robert G. Williams

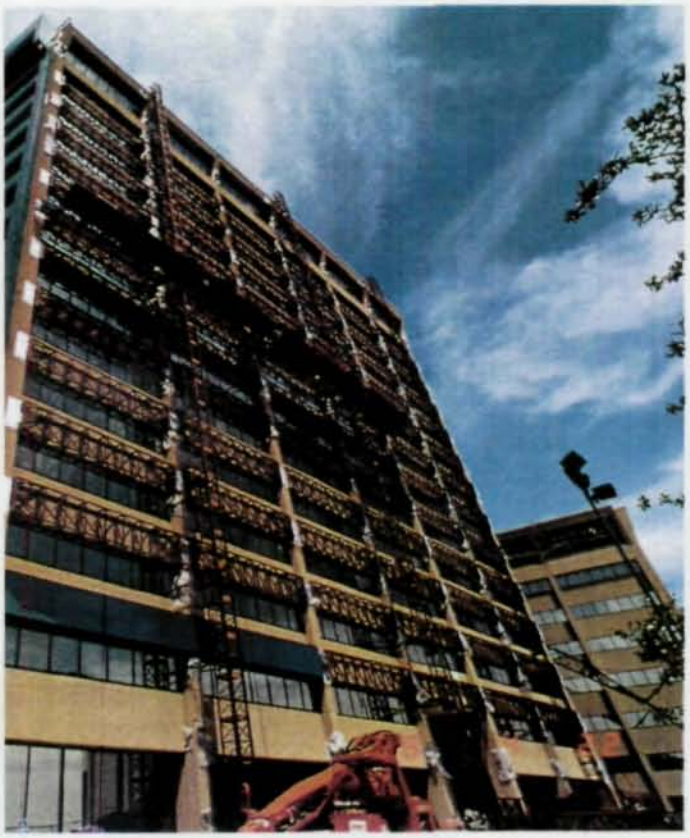
A new owner acquired a 15-story office building. Its building framework is in post-tensioned concrete beams and slabs with reinforced concrete columns. The water penetration problem through the building skin existing at the time of the ownership change had to be remedied. A design team of architect, structural engineer and contractor was formed and given the task of (1.) creating a new look to the building, (2.) stopping the penetration of water through the existing building curtain wall, and (3.) doing all this without disrupting the daily operation of building tenants who occupied the various floors.

The architect felt the "idea was to fashion a new exterior to create distinction and make more durable visually the basic character of the original thin-skinned wall system." He felt that "by furring the new wall system forward past the original column pilasters, deep window returns and sloping sills were at once created in the new, flush and sleek wall surface." His idea of the new profile is shown in the wall (see drawing).

This new profile involved moving the plane of the curtainwall forward away from the existing curtainwall, in line with the projecting pilaster-like covers at the columns. These column projections are architectural elements covering the building columns that do not project far away from the slab edges. The new facade puts the curtainwall gravity load forward at a large eccentricity from the existing curtainwall support system. A structural analysis indicated a new support system for the new profile was called for.

Building after remodeling. See "before" on p. 37 to note striking difference.





Building under construction, showing trusses being erected. Wall section of remodel (l.) details construction.

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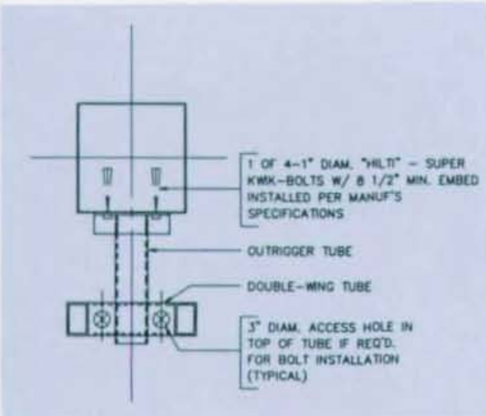
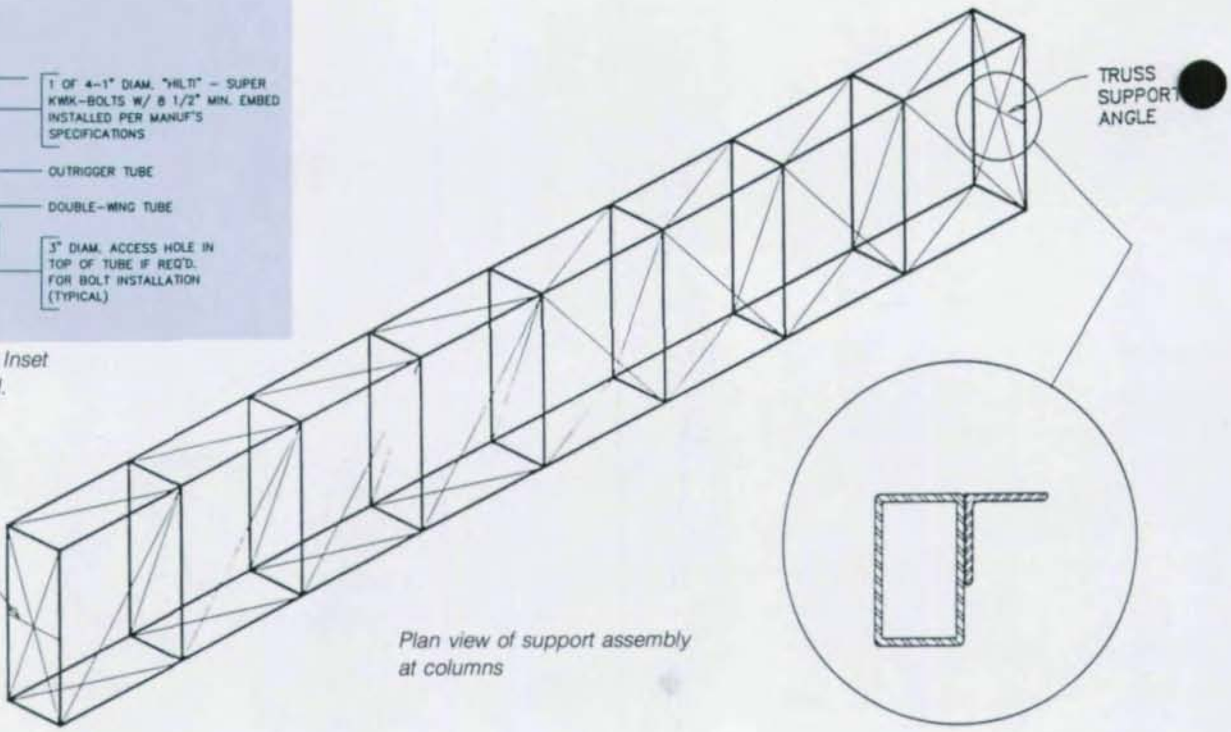


Box truss geometry. Inset shows support detail.



Plan view of support assembly at columns

Here are some of the practical requ ments:

1. The new support system should be one that could be mounted from the outside, so expensive tenant finish work need not be redone; also, tenants' businesses would not be disrupted.
2. The number of penetrations of the existing curtain wall should be kept to a minimum so as not to exacerbate the water penetration problem.
3. Since the floor structure was in post-tensioned concrete, any floor attachment devices should not damage the post-tension tendons.
4. Obviously, the new support system should be strong enough to resist both the vertical gravity loads and the horizontal wind loads and transfer these reactions to the building frame.

Several brainstorming sessions resulted in the final scheme. It was decided that welded box trusses (30 to 36 ft long, about 5 ft deep and 21 in. wide) nestled between the old and the new skin spanning between the column pilasters be erected and the new skin attached to it all from the outside. These trusses could be fabricate one piece per each bay and lifted in t thus reducing the field labor and avoiding extensive scaffolding. The contractor had

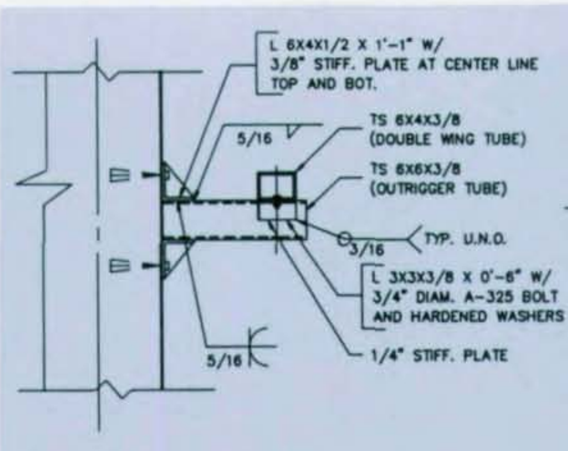
Building before *extensive skin remodeling*

a hoisting system that could climb vertically in any one bay at a time. This was used to lift the trusses to the required position. The trusses were mounted from the top floor down so those at the lower floors did not obstruct erection of those above.

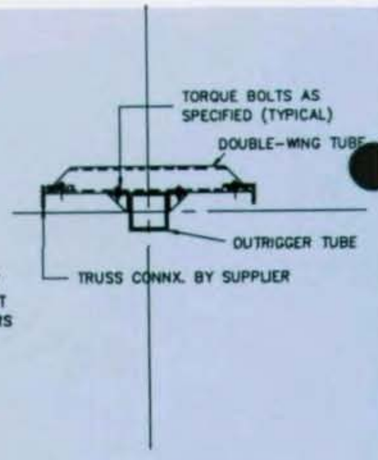
Transferring the loads from these trusses to the building columns was accomplished thusly:

1. An outrigger assembly consisting of a $6 \times 6 \times 6 \times 3/8$ tube welded to two end angles at top and bottom was attached to the concrete column with expansion bolts. This outrigger assembly was installed through a small opening in the front cover of the column pilaster.
2. A structural tube of size $6 \times 4 \times 3/8$ was horizontally inserted from the openings in the side covers and bolted on top of the outrigger $6 \times 6 \times 3/8$ tube at right angles to it, forming a "double-wing" support system for trusses from each end. The ends of this tube projected past the pilaster covers and were mitered so the truss could be "dropped in" and the support angle seated and levelled.

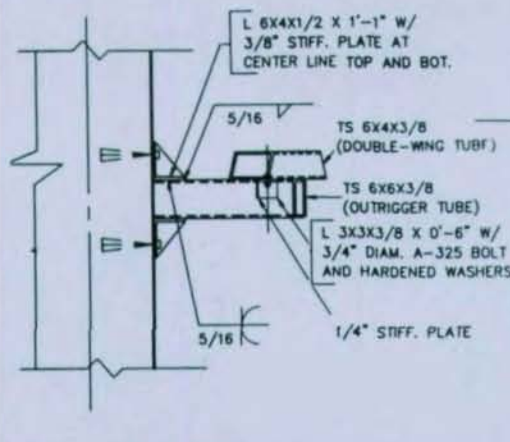
   Costly removal of the pilaster covers was thus eliminated because the box trusses could be hoisted and erected between, but clear of, the pilasters.
3. The truss support was kept near the top of the truss, to assure stability during truss erection and to prevent top roll.
4. The lateral loads from the truss were transferred to the existing curtain wall support system at selected points and to the columns with suitable connections.
5. Because the trusses were erected one bay at a time, the gravity load was transferred at only one end of the double-wing. This one-sided load induced torsion in the column outriggers. This torque and the consequent twist were



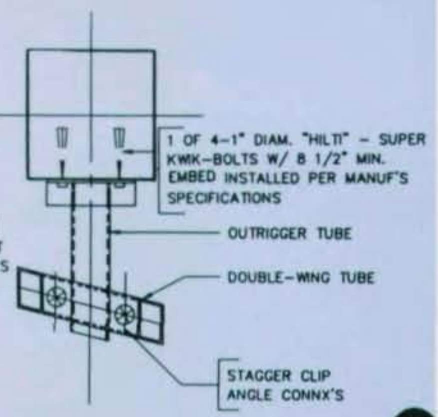
Side view of support assembly



Front end of assembly



Side view at south face columns



Support assembly at south face

considered and designed for in the selection of the member sizes and the expansion bolts.

6. At the south face of the building, where the edge of the building is at an angle, similar details were adapted as shown in the plan view and side view.

The success of this project depended on the easy and economical truss erection. Welded design of the trusses permitted the use of small and thin members, thus keeping the truss weight down. Bolted connections could have involved larger member sizes to achieve the proper edge and end distances for the bolt holes; even possibly gussetted connections could have resulted, further increasing the truss weight. Welding also contributed to the substantial stiffness of the truss system, eliminating any significant distortion during erection and preventing the "oil-canning effect" on the new exterior metal skin. Welded outrigger assembly proved torsionally stiff when truss loads were unsymmetric, the load occurring on only one end of the double-wing tube. □

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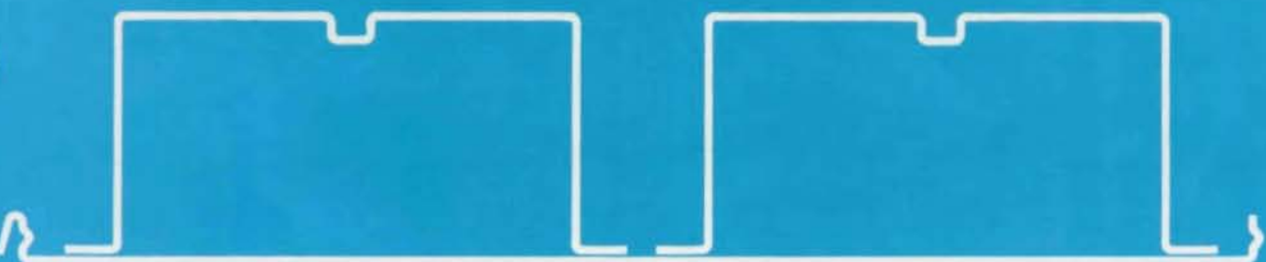

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