VOLUME XXIV NUMBER 3/THIRD QUARTER 1984

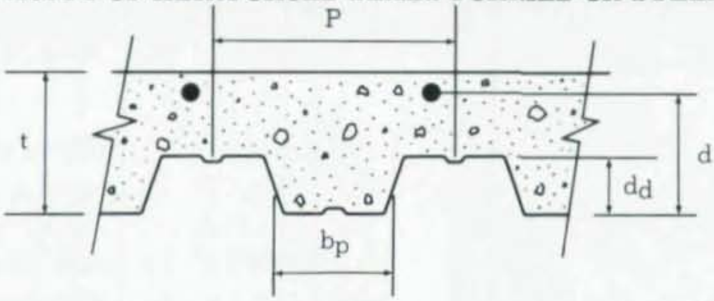
MODERN STEEL CONSTRUCTION

Skin is More Than Beauty-Deep!
Steel's Flexibility Slides to Success
Function Creates Form of Symphony Pavilion
Modular Steel Construction Speeds Schedule
How Steel was Switched to American Suppliers

DECK DESIGN DATA SHEET

NO. 5

A SIMPLE REVIEW PROCEDURE FOR DETERMINING THE NEGATIVE MOMENT CAPACITY OF REINFORCED SLABS FORMED ON STEEL DECK.



t=total slab thickness, from bottom of deck to top of slab, inches.
P=pitch of deck (center to center of ribs), inches.
d_d=depth of deck, inches
d=distance to center of reinforcing steel from bottom of deck, inches.
A_s=area of reinforcing steel (not the deck), sq. inches/ft.
b_p=average width of one rib, inches.
$b=12\,(b_p)/P$, inches per ft. of width.
Conventional reinforcement concrete design procedures apply—such as the elastic method from ACI 318-63.

$f'_c=3000$ psi, $f_c=1350$ psi, $f_s=30000$ psi, $n=9$ — example values

$P=A_s/bd$
$k=\sqrt{2\,pn+(pn)^2}-pn$
$j=1-k/3$

$M_c=\frac{1}{2}f_ckjbd^2$ (or) $M_s=A_sf_sjd$ (least value governs, inch pounds)

UNITED STEEL DECK, INC. PROFILE	w/h	b_p	P	b
1.5" Lok—Floor	3.85	6	12	6
2" Lok—Floor	3	6	12	6
3" Lok—Floor	2	6	12	6
N—Lok	0.75	2.25	8	3.375
B—Lok	1.5	2.25	6	4.5
Inverted B—Lok	2.5	3.75	6	7.5

Notes: 1.) If the deck is unshored (during the pour) the design moment does not need to include the weight of the slab.
2.) This design procedure is applicable for either composite or non-composite deck.
3.) The w/h values are used to determine stud strength if composite beams are being used. Note that N-Lok is not efficient for composite beams.

NICHOLAS J. BOURAS, INC.
P.O. BOX 662, 475 SPRINGFIELD AVE.,
SUMMIT, NEW JERSEY 07901 (201) 277-1617

clean steel by Pangborn

Engineered descaling and surface preparation systems for steel fabricators.

Now you can descale steel faster, cleaner and safer. And at the lowest possible operating and maintenance cost per ton of product cleaned.

Today, Pangborn Rotoblast® descaling is the most efficient way to clean steel products . . . plates, sheets, coils, rods, billets, slabs and rolled or fabricated structures of all types and shapes from the very large to the very small.

A Pangborn Rotoblast system can clean, prepare and provide the proper surface finish for the application of modern coating systems. Additionally, Pangborn descaling can lengthen tool and die life and improve burning, welding and machining operations.

Pangborn's horizontal roller conveyor descaler, with inverted "T" opening, handles wide flange beams up to 36" high, steel plate up to 60" wide and a variety of rolled and fabricated shapes.

Pangborn offers a full line of standard designs. Or we can custom engineer and manufacture a complete system to your special requirements . . . any size, any shape, any speed to fit into your manufacturing or fabricating process. And all of our descaling systems are built with traditional Pangborn quality, and are competitively priced.

For the most efficient, cost cutting descaling and surface preparation systems . . . talk to Pangborn first. We put over 75 years of applications engineering, manufacturing know-how and full service back-up into everything we build. Contact: Pangborn, Box 380, Hagerstown, Maryland 21740. (301) 739-3500.

Talk to the leader ...first.

MODERN STEEL CONSTRUCTION

Published by

American Institute of Steel Construction

The Wrigley Building
400 North Michigan Avenue
Chicago, Illinois 60611

REGIONAL OFFICES

NORTHEAST REGION	
New York, NY (Hdq.)	212/695-4291
Boston, MA	617/329-7417
Philadelphia, PA	609/858-9354
Pittsburgh, PA	412/443-8840
SOUTHERN REGION	
Atlanta, GA (Hdq.)	404/458-7679
Charlotte, NC	704/541-0960
Dallas, TX	214/630-5236
Houston, TX	713/270-6363
CENTRAL REGION	
Chicago, IL (Hdq.)	312/670-2400
Detroit, MI	313/352-5558
Minneapolis, MN	612/888-3791
St. Louis, MO	314/721-1332
WESTERN REGION	
Los Angeles, CA (Hdq.)	818/444-4519
Denver, CO	303/831-4622
San Francisco, CA	415/932-0909
AISC HEADQUARTERS	
Chicago, IL	312/670-2400
GOVERNMENTAL AFFAIRS	
Washington, D.C.	202/466-5548

VOLUME XXIV NUMBER 3/THIRD QUARTER 1984

CONTENTS

1985 INTERNATIONAL SYMPOSIUM SET

The program for the 1985 International Engineering Symposium on Structural Steel—which supplants this year's National Engineering Conference—will feature papers by an outstanding group of structural engineers, educators, researchers, architects and consultants from eight countries. The agenda includes experts in both theory and practice whose innovative concepts have often set the industry's standards in the design of buildings and bridges. Their contributions to the design and construction of steel structures now leads the way to future technology. See the ad on pg. 30.

EIGHT SCHOLARSHIPS TO BE AWARDED

Eight $5,000 graduate fellowships will be awarded by AISC in 1985 to graduate or architectural engineering students who study towards an advanced degree related to structural steel design. The Fellowship Awards are granted on the basis of the candidate's proposed course of study, scholastic achievement and faculty recommendation. Applications are now available at college civil or architectural engineering departments, or from the AISC Education Foundation, 400 N. Michigan, Chicago, IL 60011.

NOMINATIONS INVITED FOR T.R. HIGGINS LECTURESHIP AWARD

Applications are now being accepted for the 1985 Theodore R. Higgins Lectureship Award, which recognizes the author of the most significant engineering paper related to steel in the five-year period from Jan. 1, 1979 to Jan. 1, 1984.

The winner, who receives a $3,000 cash award, presents his paper on six occasions during 1985. A jury of six distinguished engineers from the fields of design, education and the fabricated structural steel industry selects the winning author. Nominations, which should be directed to the Committee on Education, AISC, 400 N. Michigan, Chicago, IL 60611, must be received by Nov. 16, 1984.

FOURTH ANNUAL AWARDS BANQUET DECEMBER 4

The prestigious 4th Annual Awards Banquet to honor this year's Prize Bridge Award winners will be held Dec. 4 in Chicago's Westin Hotel. A well-known panel of six jurors chose 26 of the most outstanding bridges from this year's 93 entries. The black-tie banquet, now an industry tradition, provides a forum for leading architects, structural engineers, bridge designers, contractors and fabricators.

Highlight of the evening will be a multi-media presentation by the City of Baltimore on the planning, design and construction of the Ft. McHenry Tunnel. Dallas E. Wiegel, director of Baltimore's Community Affairs Dept. will present the show. Tickets are available from AISC Public Affairs, Chicago. 312/670-2400.

numerous special features were developed.

Massing consists of a 54-story tower and a 17-story "bustle" (attached low-rise portion) of similar appearance, to echo H.H. Richardson's landmark Allegheny County Courthouse and tower. The use of solid plate panels with only approximately 25% glazing gives a visual weight similar to the surrounding traditional buildings, while aiding energy conservation.

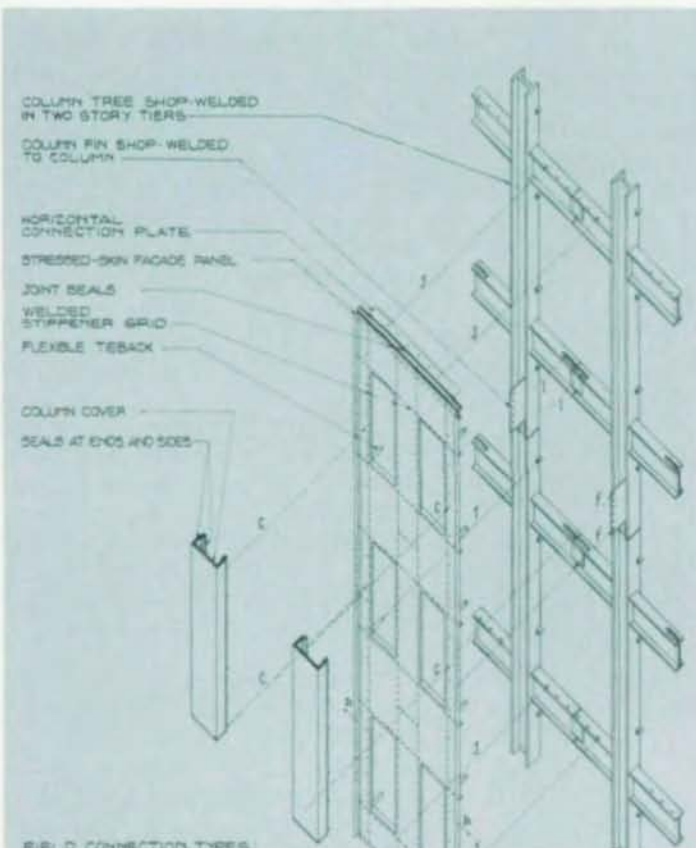
The tower and bustle have sloping upper walls similar to the mansard roof of the Union Trust Building. Start of the bustle slope coincides with start of the Union Trust roof. Setbacks also occur at building tops to further define their special nature. Tall, narrow windows and protruding column covers create a strong sense of verticality similar to the courthouse tower, and serve to distinguish it from the broader U.S. Steel Building. An elongated octagonal floor plan creates eight relatively narrow faces, further diminishing its apparent width.

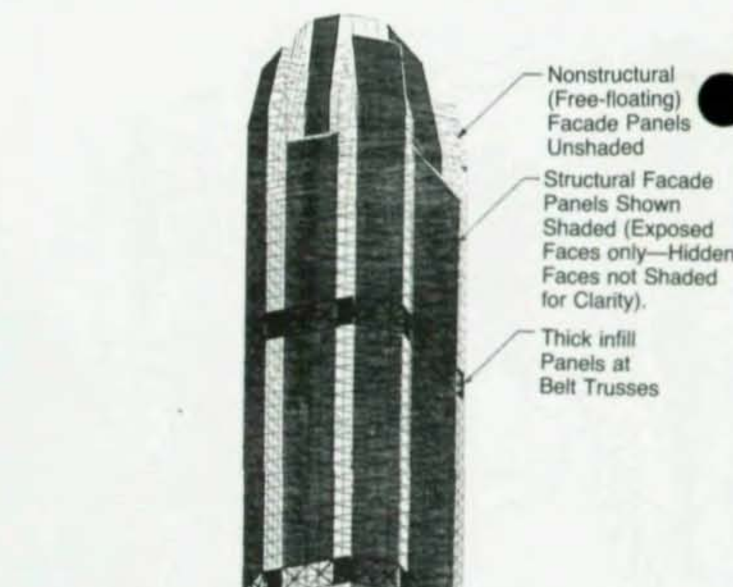
Corner projections, which create eight desirable bay-window corners per floor, reflect the thickened corners of the courthouse tower. The finish colors, two shades of warm gray-to-beige, harmonize with the gray granite courthouse and Union Trust buildings, the rich brown Cor-ten U.S. Steel Building and the red brick William Penn Hotel.

Structural Concepts

Because of the building height, desired clear spans, local construction market and developer preference, steel framing was chosen for this project. Several wind-resisting systems were considered. Core bracing proved uneconomical because the core is quite narrow, giving the bracing system a very slender 20:1 aspect ratio. It would require excessive additional steel in core columns to control drift, and large uplift forces requiring expensive foundations. Core bracing, plus outriggers, would take useable space from outrigger floors, and require that core columns and perimeter columns align, compromising core efficiency. Full-width moment frames also require alignment and would need deep beams to develop adequate stiffness across the long beam spans. This would force an increase in floor-to-floor height from the desired 12 ft, and would cause the core to grow to accommodate wider beams. And braced tube schemes, using multistory X or V braces, were incompatible with the architectural design.

The framed-tube scheme finally selected proved structurally efficient and gave the architect freedom to design an extremely compact, efficient core. Framed tubes conventionally use wide, closely spaced columns and deep, stiff spandrels to form a nearly solid tube or cylinder which resists lateral forces by cantilevering up from the foundation. The spandrels are usually upset, projecting above the floor to provide adequate stiffness to limit building drift. But here, with cooperation between the architect and engineer, and the developer, the facade was designed to add its stiffness to that of the framed tube. Thus the tube could be sized just for strength. When designing for strength alone, the columns became narrow W14's on a 10-ft spacing (A572 Gr. 50 steel), and spandrels are W24 to W30's (A36 steel), which fit beneath the floor. This freed up useable floor space all along the perimeter, but wind drift or sway at the top floor





Stressed skin-tube interface (l.); and tower model with facade panels (above).

numerous special features were developed.

Massing consists of a 54-story tower and a 17-story "bustle" (attached low-rise portion) of similar appearance, to echo H.H. Richardson's landmark Allegheny County Courthouse and tower. The use of solid plate panels with only approximately 25% glazing gives a visual weight similar to the surrounding traditional buildings, while aiding energy conservation.

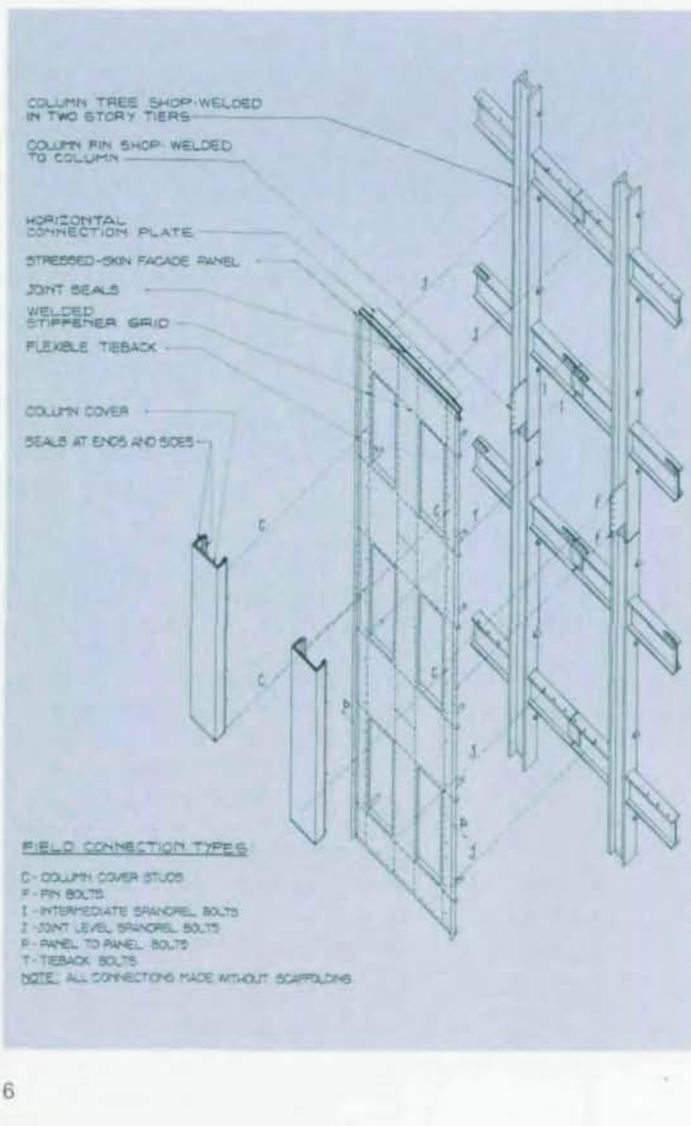
The tower and bustle have sloping upper walls similar to the mansard roof of the Union Trust Building. Start of the bustle slope coincides with start of the Union Trust roof. Setbacks also occur at building tops to further define their special nature. Tall, narrow windows and protruding column covers create a strong sense of verticality similar to the courthouse tower, and serve to distinguish it from the broader U.S. Steel Building. An elongated octagonal floor plan creates eight relatively narrow faces, further diminishing its apparent width.

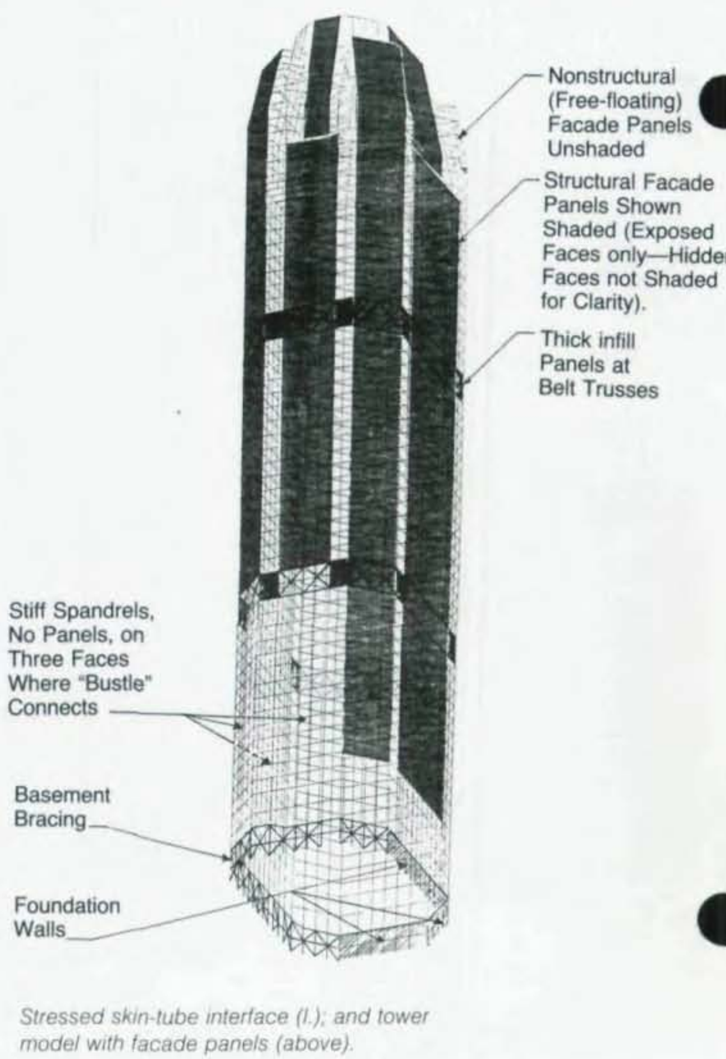
Corner projections, which create eight desirable bay-window corners per floor, reflect the thickened corners of the courthouse tower. The finish colors, two shades of warm gray-to-beige, harmonize with the gray granite courthouse and Union Trust buildings, the rich brown Cor-ten U.S. Steel Building and the red brick William Penn Hotel.

Structural Concepts

Because of the building height, desired clear spans, local construction market and developer preference, steel framing was chosen for this project. Several wind-resisting systems were considered. Core bracing proved uneconomical because the core is quite narrow, giving the bracing system a very slender 20:1 aspect ratio. It would require excessive additional steel in core columns to control drift, and large uplift forces requiring expensive foundations. Core bracing, plus outriggers, would take useable space from outrigger floors, and require that core columns and perimeter columns align, compromising core efficiency. Full-width moment frames also require alignment and would need deep beams to develop adequate stiffness across the long beam spans. This would force an increase in floor-to-floor height from the desired 12 ft, and would cause the core to grow to accommodate wider beams. And braced tube schemes, using multistory X or V braces, were incompatible with the architectural design.

The framed-tube scheme finally selected proved structurally efficient and gave the architect freedom to design an extremely compact, efficient core. Framed tubes conventionally use wide, closely spaced columns and deep, stiff spandrels to form a nearly solid tube or cylinder which resists lateral forces by cantilevering up from the foundation. The spandrels are usually upset, projecting above the floor to provide adequate stiffness to limit building drift. But here, with cooperation between the architect and engineer, and the developer, the facade was designed to add its stiffness to that of the framed tube. Thus the tube could be sized just for strength. When designing for strength alone, the columns became narrow W14's on a 10-ft spacing (A572 Gr. 50 steel), and spandrels are W24 to W30's (A36 steel), which fit beneath the floor. This freed up useable floor space all along the perimeter, but wind drift or sway at the top floor





Stressed skin-tube interface (l.); and tower model with facade panels (above).

of this relatively flexible tube would have been an unacceptable 30-in. or 1/290th of height. By using the steel plate facade as infill, the frame was stiffened and wind drift became an acceptable 15 in. or 1/590th of height. Because the facade is used only for deflection control, not for strength, it does not need fireproofing or flame shields.

Facade Analysis

Although required only for stiffness, the facade panels need sufficient strength to resist the forces imposed. It was necessary to keep working stress below buckling levels to provide good stiffness, avoid a rippling appearance and possible glazing problems. To accomplish this, panels were analyzed in several ways:

First, the small rectangular subpanels defined by the stiffener grid were analyzed using classical plate buckling theory to establish critical buckling stresses, assuming that the stiffeners are fully effective. Stiffener effectiveness was checked by using the AISI specification for design of cold-formed laterally unbraced beams (since panel proportions made it a thin-wall section). Stiffener and subpanel buckling was also checked by a NASTRAN buckling program.

Second, a fine-mesh finite element model of the entire panel with appropriate edge restraints was used to find the pattern of working stresses expected at various levels of panel shear, and to determine the lateral spring constant of the panel.

Third, the elastic buckling stresses found in Step 1 were converted to allowable stresses, with due consideration for inelastic behavior. The slenderness ratio L/r, which corresponds to the elastic buckling stress, was found, and that L/r was used in the usual AISC compression formulas for allowable stress.

Fourth, a full-size, three-story steel prototype panel was fabricated by USS Fabrication and tested at Fritz Laboratory of Lehigh University. Overall panel stiffness and buckling strength was in good agreement with the analysis.

At this point, a hierarchy of panel types versus shear force was established, starting at ¼-in. plate with the basic stiffener grid, adding stiffeners and/or plate thickness, and ending at 5/16-in. plate with all spandrel and mullion subpanels divided by additional stiffeners. This hierarchy was used to develop a panel schedule similar to a column schedule, since the overall building computer runs showed shears varying from face-to-face and floor-to-floor.

Facade Detailing

Typical panels are three stories high by 10-ft wide to minimize joints, yet permit easy shipping. They consist of ¼-in. to 5/16-in. A36 face plates, a grid of 4-in., 5-in. and 6-in. A36 stiffeners aligned with window edges, and bent plates or angles at panel edges. Because panel installation closely followed frame erection, the panels required isolation from the column shortening which would occur as erection proceeded. Thermal expansion also had to be relieved. These goals are met by a unique attachment system which limits panel-to-frame connections to vertical fins at panel mid-height, flexible horizontal plates at panel top and bottom, and flexible tiebacks at window heads and sills.

Panel-to-panel connections are used to

Erection of column trees at sloping top. Note fins for panel connections. Web holes give bolting access at panel horizontal joints.

Column cover butts cover below and studs protrude through slots in panel. Note bolts at panel edges.

ensure that each building face acts as a true stressed skin. Bolted connections at top and bottom transfer the horizontal wind shear from upper to lower panels. These connections attach to the spandrel beam to bring in additional shear from wind on that floor, and compress two lines of closed-cell neoprene which seal the horizontal shiplap joint against weather. Panel-to-panel stitch bolts along vertical edge angles transfer wind shear between adjacent panels. By concentrating bolts in the areas adjacent to window openings, the two narrow "mullion" strips on adjacent panels are forced to act as one wider mullion with much greater stiffness and strength.

At vertical fins, fin-to-column connections need to transfer only the *change* in shear between adjacent panels. Interestingly, while the largest shear in panels occurs when panels act as part of the tube web, the largest *change* in shear between panels occurs on the tube flanges. Where building faces end, the stressed skin is interrupted by bay-window corners, and a heavy vertical edge angle serves to collect all the panel shear and deliver it to a special, large vertical fin on the building frame. Tiebacks provide resistance against wind normal pressures and hold the grid against buckling.

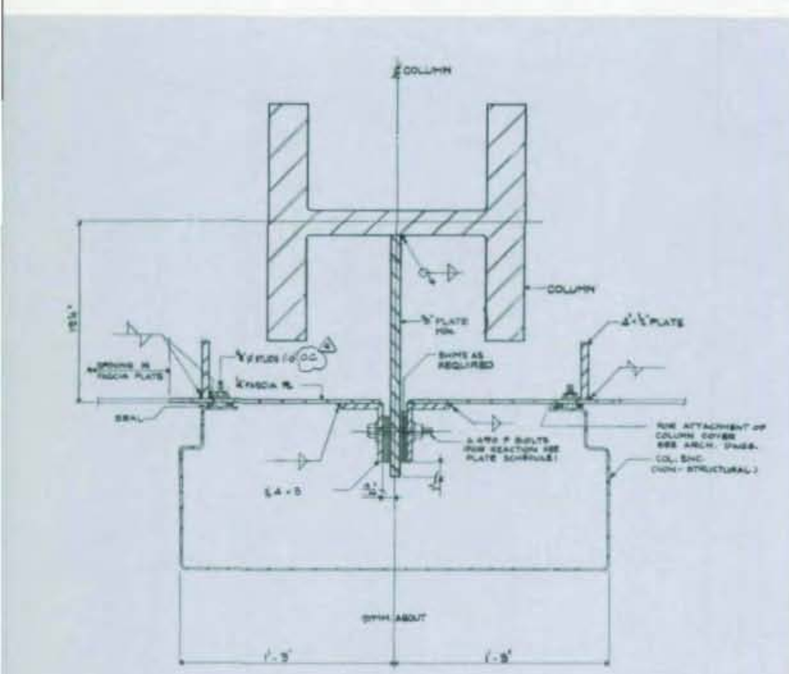
While panel horizontal joints are easily sealed, the numerous bolts and shims at vertical edges require a different approach. The column covers, brake-bent from 12-ft lengths of ⅛-in. steel plate, cover this joint and are sealed against the panels with closed-cell neoprene strips. These are compressed by ½-in. threaded studs which poke inward through the panels. Butting ends are sealed by neoprene held between end diaphragms of the stacked covers. To avoid a chimney effect in the covers, one end diaphragm every few floors is extended to block virtually the full cross section of the cover. To permit weeping of any water which may enter, the bottom seal at the base flashing is an open-cell sponge.

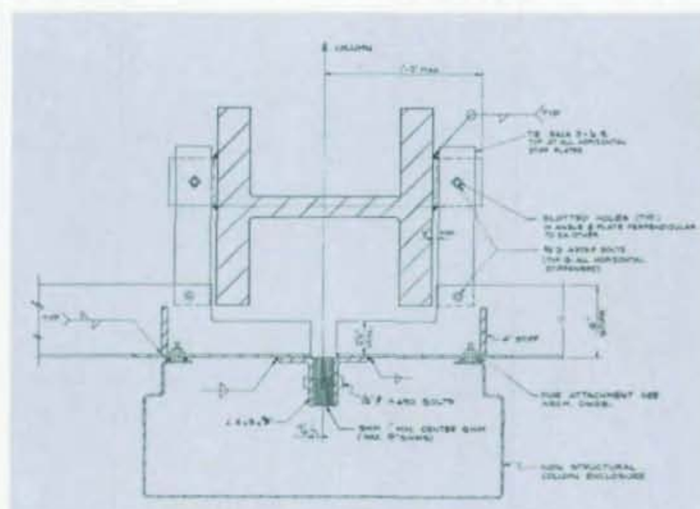
Where column covers bridge across panel joints, and where non-structural panels occur, such as at the flexible bay-window corner panels, motion is permitted by the use of slotted holes for studs and pairs of ultrahigh molecular weight polyethylene slip pads under plate washers.

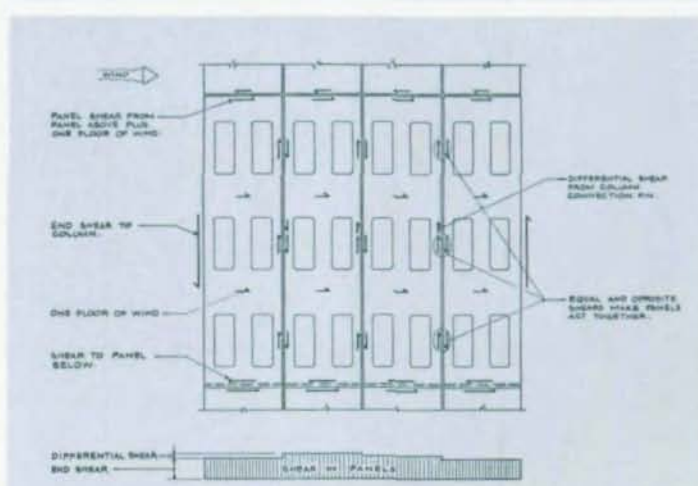
Panel Fabrication

Because 6,300 tons of steel are in the skin, facade fabrication used as much standardization as possible. Face plates were ordered oversized to ensure unmarred final edges and true final shapes. Window openings were cut and edges trimmed to true size on a numeric-controlled plasma torch table. Window openings have corner bulges to provide a stress-reducing radius without interfering with the glazing, and the n-c torch easily provided this potentially tricky detail. Jigs were used to fabricate stiffener grids square and to a true plane. Double fillet welds provide continuity through stiffener intersections. Then the face and the grid were connected, using ⅛-in. intermittent staggered welds. This weld size was found to limit oil-canning on ¼-in. plates to acceptable limits, and to show virtually no oil-canning on 5/16-in. plates. Edge angles and bent plates were fillet-welded last.

The panels received a shop-applied four-part treatment consisting of sandblasting, zinc-rich primer, epoxy interme-







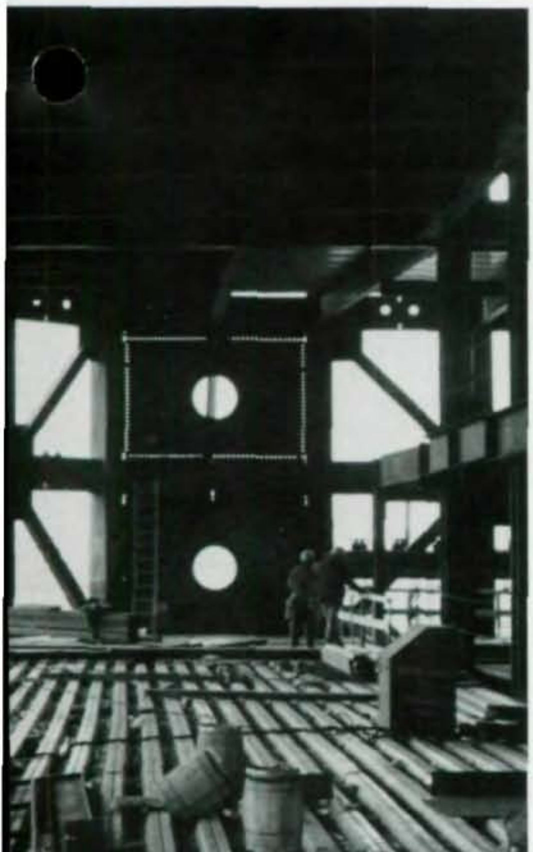
Plan section at panel-to-column connection fin (above). Forces acting on panels (r.). Section at panel-to-panel vertical joints (r.).

diate and two coats of urethane finish. Insulation was also shop-applied using impaling pins, to permit quick erection directly from the delivery truck. Transportation costs were minimized by using jigs, which permitted stacking of several panels on a trailer, yet protecting their finish from damage. After adjustment and bolting up, insulating glazing was field-installed from inside using neoprene zipper gaskets in a bed of uncured butyl sealant. The inner light of glazing is supported by a PVC weep evaporation channel on the sill stiffener.

Special Features of Structural Design

In addition to the use of a stressed-skin facade, the 21,000-ton steel frame incorporated numerous other special features:

- A two-step tower analysis. One tower model considered the frame only, and included 3,000 nodes, 6,000 members and 9,200 degrees of freedom. A second tower model had facade panels added, simulated by 1/20-in. thick steel membranes. Each member was checked against forces in both models resulting from 70 mph fastest-mile winds. Panel shear forces were derived directly from the second model.
- Explicit recognition of the P-delta effect. When a building drifts, the total of column loads at a story P times the interstory drift, delta, adds to wind moment. P times delta divided by story height is the additional story shear generated. For the rather flexible frame-only case, P-delta typically added 17% to wind shears and moments.
- Eight bay-window corners on each tower floor, which interrupt the line of structural facade panels and standard tube bays. Even with very heavy W33 or built-up beams, these weak links caused the framed tube to act too much like a collection of individual walls. To reconstitute proper tube action, infill plates 1½-in. thick and belt trusses were required at the 17th and 35th levels, which are mechanical floors.
- Box columns at 45° corners. Because of the nature and orientation of wind moments acting on these columns, internal stiffeners required welding on all four edges. Three edges were welded before closing the box, and the fourth edge was welded by the electroslag process through a keyhole in the box.
- Free circulation between the low-rise bustle and the tower. Built-up spandrels compensate for the loss of facade panel stiffness in these areas, to avoid tower twist.
- Tuned bustle stiffness. An expansion joint between tower and bustle would create more problems than it would solve. To help the bustle move together with the tower, a line of internal chevron bracing was provided and sized to give stiffness compatible with the tower.
- Vierendeel column pickups: 23 of the 28 perimeter bristle columns are interrupted for lobbies, driveways, loading docks etc. The spandrel-and-column grid on upper floors acts to carry most of the load by Vierendeel action, permitting relatively light pickup girders.
- Shallow floor-to-floor height (12-ft) with an 8 ft-6 in. ceiling and a 45-ft span. Unshored composite W24 beams support an electrified 4½-in. slab on a 2-in. composite metal deck. Shop-cut openings pass mechanical services. An inhouse computer program determined reinforcement requirements, so only 40% of the 4,000 beam cuts required reinforcement. Such reinforcement is generally limited to a pair of bars at the bottom edge of the hole.



Belt trusses on faces and infill plates at corners occur at 17th (shown) and 35th-floor mechanical rooms to improve tube action.

- Offset core columns which project into elevator lobbies, rather than into the circulation corridor around the core. For a given corridor clear width, this gives an additional band of useable space all around the core.
- Standardized slotted beam seats for skewed beams. By skewing, framing is direct and easy to erect while still permitting freedom of core column placement.
- Cambered core columns. Because vertical load plus wind moment governs perimeter columns, they act at a lower stress than the vertical-load-only core columns. Core columns were fabricated about 1/20-in. longer per story (1/8-in. in three out of four two-story tiers) to provide level, not dished, floors when all dead loads including permanent walls are in place.
- Minimum-sized core columns. At lower levels, columns are virtually solid A572 Gr. 42 steel consisting of side-by-side plates welded together.
- Grillage footings at core columns. To avoid overloading weak claystone underlying the sandstone bearing stratum, the bearing level had to be kept high. Conventional reinforced concrete footings would be too thin and fail in punching shear. Each steel grillage consists of a bottom tier of 15 to 16 S24 sections and a top tier of three W14 jumbo sections, all with pipe and strap separators. Bottom tier beams were set shimmed on a mudslab to give a level top plane. Top tier beams were shop milled to have a level top bearing plane to receive the base plate when they rest on a level surface. The assembly was then encased in concrete.
- Foundation bracing. Half of the tower perimeter is incorporated within the foundation wall, which is designed to resist the wind shear, and to act as a hold-down beam for those corner columns which may experience uplift. To avoid tower twisting and limit drift at street level, the other perimeter faces have heavy steel wind shear bracing from foundation to street level.
- Easy retrofit for the bank vaults, mezzanines, stone finishes and grand stairs requested by tenants, because of the adaptable nature of steel framing.

At One Mellon Bank Center, the marriage of a stressed-skin facade to a framed tube creates a distinctive new structural system which is practical, efficient and economical. It should prove to be a useful addition to the family of high-rise wind resisting schemes. □

Top of "bustle" shows crisp details and numerous planes of stressed-skin facade.

A paper on One Mellon Center won the $5,000 Gold Award in the James F. Lincoln Arc Welding Foundation's 1983 Welded Design and Engineering Award Program.

Architect
Welton Becket Associates
New York, New York

Structural Engineer
Lev Zetlin Associates, Inc.
New York, New York

General Contractor
Turner Construction Company
Pittsburgh, Pennsylvania

Steel Fabricators
American Bridge Division, U.S. Steel Corp.
Montague-Betts
Pittsburgh Bridge & Iron

Panel Fabricator
Calumet Industries Works Incorporated
Gary, Indiana

Steel and Panel Erector
American Bridge Division, U.S. Steel Corp.
Pittsburgh, Pennsylvania

Owner
500 Grant Street Associates
Pittsburgh, Pennsylvania

Developer
U.S. Steel Realty Development
Pittsburgh, Pennsylvania

When's the last time you asked for a custom hot-rolled section?

WBC Custom-Welded Sections Beat Hot-Rolled Beams Six Ways.

1. Custom Sections
It's no longer necessary to restrict your beam designs to standard handbook WF sections. Special offset or "Z" sections like these are easily produced by Welded Beam Company to suit your exact geometry, loading, and length requirements. You can even use dissimilar steels for web and flanges.

2. Reduced Fabricating Cost
Since WBC "CustomBeams" are fabricated exactly to your requirements, labor and scrap from cutting up standard WF beams to suit the job is eliminated.

3. Stronger
High Frequency welding produces a true forge weld as strong as the parent metal with no filler metal or cast structure. And, you can specify HSLA steels up to 80,000 psi yield for all or part of the beam providing greater load-carrying capability in a smaller, lighter beam.

4. Easier Assembly
Draft angles are nonexistent for WBC HF-welded beams. So flanges are flat and edges are square, simplifying fit-up at assembly. Also, beams can be delivered to exact customer lengths saving on splicing or cutting on-site.

5. Consistent Quality
Our beams are produced continuously at speeds up to 200 feet per minute on our modern high frequency weld mill. Low waste, high speed, and specification-matching controls combine to assure a consistent high-quality product.

6. Delivery Flexibility
Our mill is a more flexible manufacturing system than the traditional hot mill. As a result, shorter runs are easily achieved that shorten your lead time and let you match delivery to construction dates.

Welded Beam Company "CustomBeams" can make a dramatic difference in the integrity, scheduling and fabricating costs of your fabricated metal product.
Contact WBC now to get the full particulars.
Post Office Box 280, Perry, Ohio 44081. Telephone: (216) 259-4500.

WELDED BEAM COMPANY

Atlantis, The Water Kingdom: Steel's Flexibility Slides to Success

Sixty-five acres of water slides! The magnitude of Atlantis, the Water Kingdom in Hollywood, Fla. makes its overall design concept unique. According to the park's designer, the water-oriented theme park is the largest of its kind in the world. The $16.5-million park offers a wide array of water-related rides, attractions, leisure activities and family entertainment just off Route I-95 in east-central Florida. Figure 1 details the overall concept of the huge project.

The intricate design of the steel structural system that supports an array of water slides (flumes) from a steel tower is shown in the area circled on Fig. 1. When finally completed, the project will have 18 slides projecting from the tower at 34-ft, 44-ft, 54-ft, 64-ft and 74-ft levels. At full capacity, 4.9 million gallons of water daily will rush down the slides, enough water to support a small city. Twelve slides were built in Phase 1—nine standard 4-ft slides, one tube and two speed slides—the largest slide system ever built.

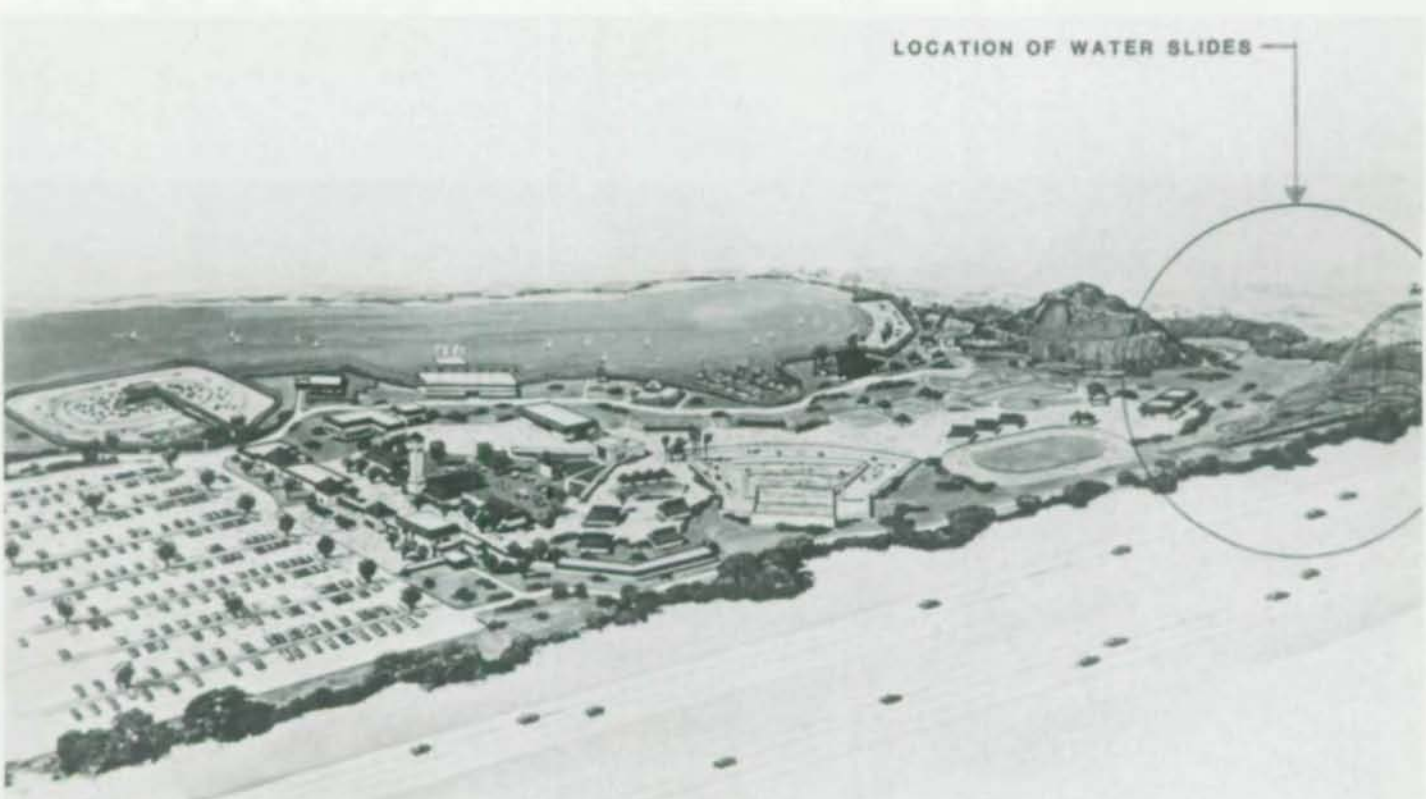
Design of the flume support systems demanded high-strength steel. The connections had to be simple because of the complexity involved when the structural members were interacting. Welding was chosen to give simplicity and a clean appearance to all the connections. The support system was also designed so the majority of the welded connections could be fabricated in the shop for economy and quality control.

Flume Structural Support System

The architect was challenged to design a system that could support a number of water slides at different heights. Figure 2 shows the problem was complicated by the unusual weaving and looping patterns of the water slides. A flume run consists of many fiberglass pieces bolted to one another. Every flume joint has an x-y-z coordinate. These unusual patterns demanded a very flexible system to support all the flume joints. As a result, the flume tree support and the trestle support were designed with the idea to carry as many flume joints as possible with the least amount of ground supports.

To keep the overall support system looking clean, a single pipe support carries a number of joints. This flume tree system proved to be more efficient than a trestle

Figure 1. Rendering of total Atlantis project site, with water slide location circled.



support (a framework of tubular steel members) at every joint. There was approximately a 30% savings in cost of steel pipe using the free design vs. an equivalent height trestle. The support systems (as high as 74 ft) had to be designed for hurricane winds up to 120 mph (required by the South Florida Building Code). Steel, chosen for its high strength and practical application, made the overall system very flexible in adapting to the unusual shape and height patterns of the slides. The same strength and flexibility were maintained by using only welded connections. Connections were simplified, thus producing a clean transition between structural members.

Flume Tree Support

The concept in designing the tree structure was to support as many flume joints as possible. The various structural components of this system are labeled in Fig.3. It was advantageous to place a pipe support at the center of the flume loop or radius where possible. This gave the system the flexibility to support a greater number of flume joints, and thus eliminated many trestles. Where there are a number of tree supports, it was most cost-effective to fabricate sets of arm systems (main arm and kick arm) of one length. The flume tree employs all-welded connections.

The mounting bracket connects the fiberglass flume to the secondary arm support (Fig. 3). The secondary arm provides enough play in the system so a proper flume fitting can be achieved before field-welding the arm. A tubular structure was chosen for the arm systems primarily for its rigidity in bending and torsion.

The applied loads considered on these members were:

- Dead load of the flume system and live loads due to people and water
- Centrifugal forces developed when a mass of people and water travels through a flume loop or radius
- Turbulence induced by high winds flowing through the spaghetti-like flume structure

The tubes also provided simple connections. The short and long arm system reduced material cost. The main arm has a maximum length of 18 ft, but in special cases it reaches out as far as 21 ft (see Fig. 4). This arm system required plate stiffeners to strengthen the pipe walls. Welding made it possible to have clean connections between the tubes and pipe while maintaining the structural integrity of the system. Since they have the same outline as the connections, the welds were more efficient in resisting a torsional load—the same efficiency for using tube members for torsion.

The tube-to-pipe connection became more complicated when two or more arm systems combined on the same plate stiffeners. A pipe jig and hydraulic jack were used for the shop fabrication of this set up. In many cases, the arm systems were interfering with each other. The problem was complicated as the number of arms interfering increased. Such interferences not only reduced the cross sectional areas of the tubes, but also introduced additional torsional stresses. Welding the entire splice was the only practical solution, which developed the full load on the connection without any additional plates. A clean connection (Fig. 5) was still maintained after being primed in the shop.

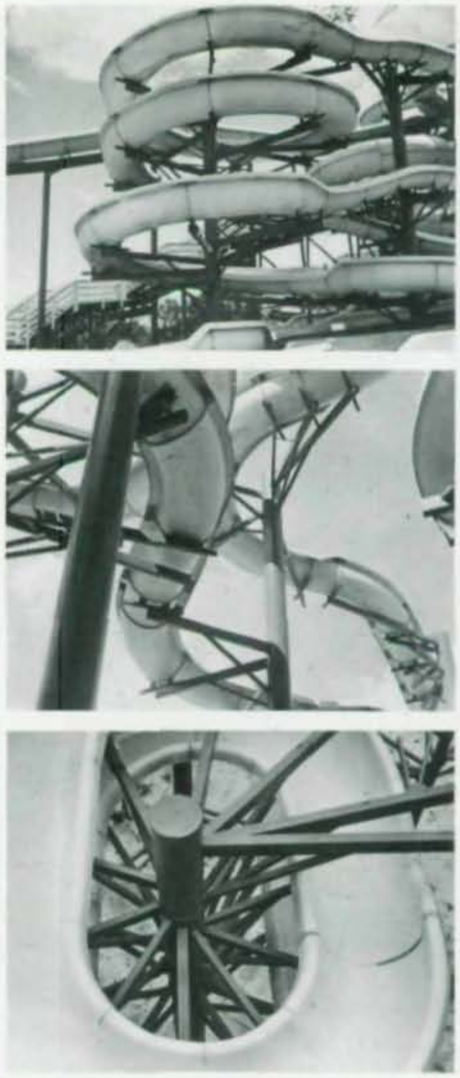
Each arm system had to be precisely welded to match an x-y-z coordinate of a flume joint. It was necessary to position the arm systems correctly, and quality welding was required. Fabricating the arm systems in the shop made this possible, and the arms could then be attached and field-welded to the pipes. Erection bolts were welded to the pipes to accurately position the arm systems.

The pipe support was the largest structural member of the tree system. A pipe was chosen because of its efficiency in resisting a lateral force, with five different size pipe columns shown in the columns schedule. In some cases, the resisting wind moment near the base was so great that the largest available size pipe had to

Figure 2. Unusual weaving and interlooping of slides complicated design problems.

be reinforced with a pipe liner. Figure 6 shows a Pipe "24" STD. (½ in. thick) lined with a Pipe "24" XS (⅜ in. thick). The bottom was cut at an angle for ease of welding the pipe(s) to the base plate. The liner, which only extends as far as structurally required (up to 15 ft), is a pipe with a single cut along its length. Plates with pre-drilled holes were temporarily welded to the pipe on each side of the cut to provide anchorage for a threaded stud with double nuts at each end. The sleeve pipe was gradually pried open by screwing the appropriate nuts. A come-a-long was then used to slip the sleeve over the pipe support. After positioning, the nuts were unscrewed, causing the sleeve to rebound to its original shape. This rebound caused the sleeve pipe to physically grip around the pipe support, giving a very tight connection to weld. Welding made it possible for the liner and pipe to perform as a single structural member. The final product reveals a liner which is unnoticeable from a distance.

It was also economical to section a long pipe into different thicknesses. The lengths of pipes to be spliced had to be determined by balancing the savings of material of a thinner member vs. the labor cost of the splice weld itself. If the ½-in. pipe was more than 13 ft above the point where a ⅜-in pipe was structurally adequate, it was more economical to weld



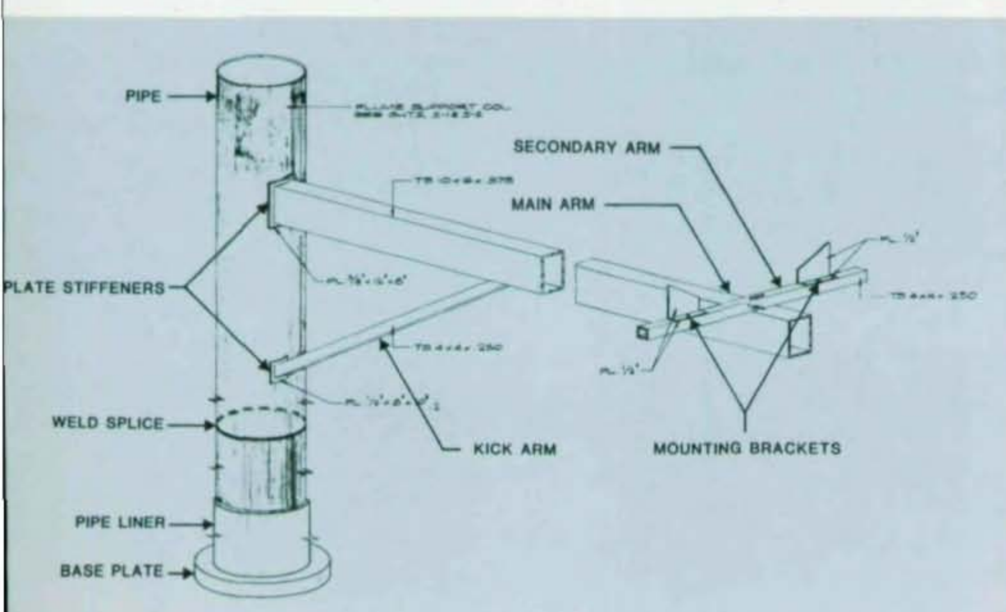


Figure 3. Flume support isometric

Figure 4 (top): Steel arm reaches out to 21-ft maximum. Figure 5 (c.): Clean connection was maintained after shop priming. Figure 6 (bott.): Final product of welded connection.

splice the ⅜-in. pipe to complete the tree. Keeping this in mind, the weld splices were located where they were cost effective. A clean transition was once again maintained throughout the pipe with the welded splice connections.

For erection purposes, it was decided to first anchor the base plate into the footing and then field-weld (a groove weld with an inner shield) the pipe(s) to the plate. This procedure provides a faster and more efficient weld, and produces a very clean connection without sacrificing the structural integrity of the system.

Trestles

The other alternative for transmitting loads was the steel trestle support. The trestles were placed on a straight flume run where the tree support was not practical, or at places beyond the arm systems' reach. This support system consists of tubular members welded to each other, with the trestles ranging in height from one to 69 ft. Some trestles were a combination of a truss and frame system, necessary because more than one flume had to be supported at different levels by the same trestle. The trestles also could be fabricated in one piece in the shop.

This project is an excellent example of the versatility possible in welding steel. Some of the attractive structural and architectural characteristics of welding are:

- It is rigid, yet ductile in strength.
- It produces a variety of connections because it can form around any shaped member.
- It produces a very clean connection, making the overall project pleasant to the eye.

Welding also permits the option of making cost-effective decisions. Substantial savings can be made if the designer realizes the potentials welding has to offer. The quality in welding has reached such a point that the structural engineer should feel quite comfortable in using it as one of his many design tools. □

Design/Structural Engineer
D. E. Britt Associates, Inc.
Fort Lauderdale, Florida

Steel Fabricator
Steel Fabricators, Inc.
Fort Lauderdale, Florida

Contractor (water slide)
Water Parks of America
Myrtle Beach, South Carolina

Owner
Six Flags Atlantis, The Water Kingdom
Hollywood, Florida

Cincinnati Symphony Pavilion: Function Creates its Form

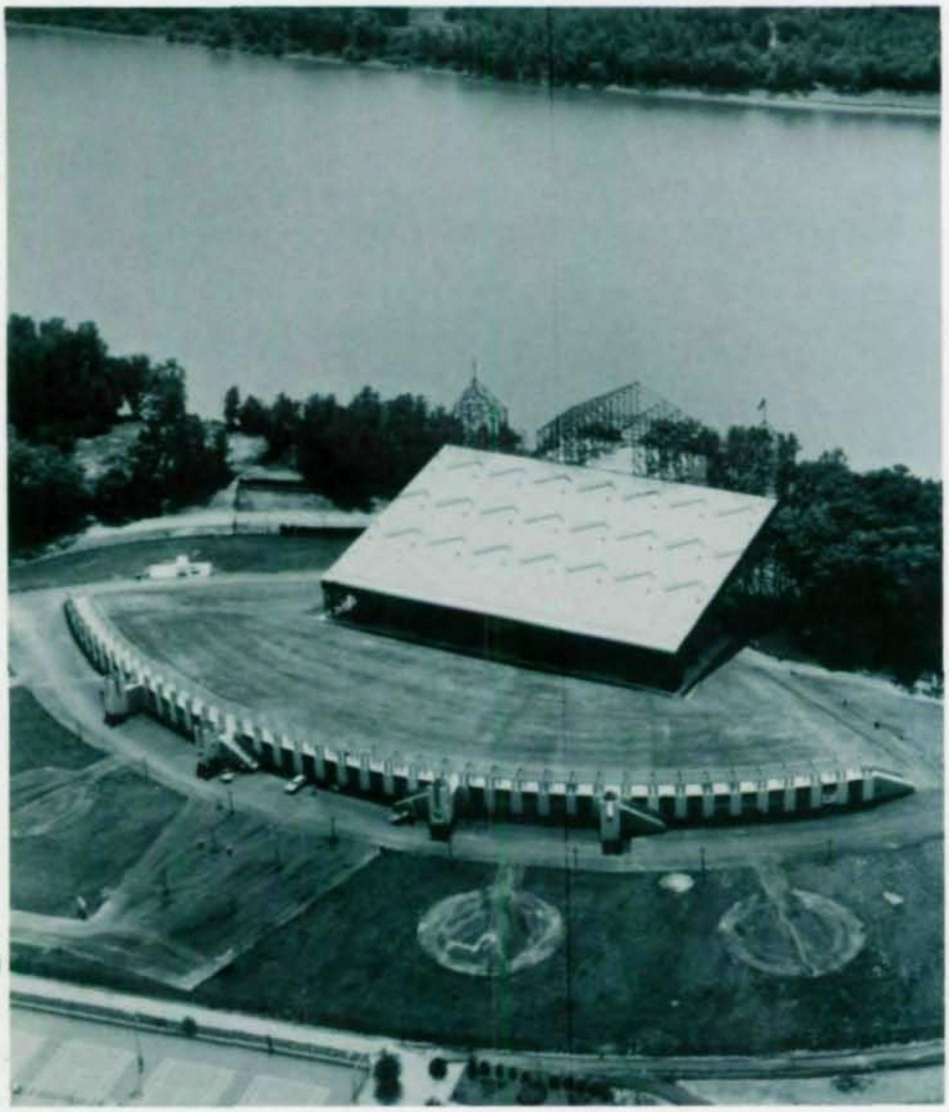
by James E. Chaplin

A new summer home for the Cincinnati Symphony has risen on the banks of the Ohio River in Hamilton Co., O.—just in time for a July 4 opening performance.

Unlike pavilions in other areas of the country, Cincinnati Symphony's new abode incorporates an enclosed stage house 110 ft wide by 55 ft deep. The stage house, poured concrete with foot-thick walls, is covered by a watertight roof. The proscenium opening is 70 ft wide by 30 ft high. At the rear of the stage house is a two-level structure that houses a green room, dressing rooms and space for instrument storage.

Long-span roof trusses 10-ft deep that span 176 ft provide a one-acre covered, open-sided seating area for music lovers. More than 4,000 can be seated in the covered area, and space on the outdoor slopes provides for another 10,000 to view the performances through the open sides and rear of the covered seating area. Since the 264-ft width of the seating area is greater than the stage house, the roof trusses are supported at either side on 33-ft square steel towers. The space between these towers and the stage house is spanned by a box truss which supports the roof trusses spaced 11 ft o.c.

Although the pavilion's location on the Ohio River is quite scenic, it poses problems. Spring floods frequently cause the river to rise to 10 ft above the stage level. Equipment, and the dressing room and stage areas, are set up to be removed and stored during the winter. The structure is permanent, but had to be built sufficiently rugged to resist damage from the water, floating objects, ice and the assaults of nature. Equipment items which cannot be readily removed and replaced (cooling



New summer home for Cincinnati Symphony at former playland site on Ohio River. Pavilion theme was orchestrated around the durability and symmetry of structural steel framing.

James E. Chaplin is vice president of the consulting engineering firm of DeSimone, Chaplin and Associates PC, New York, New York.

towers, transformers, etc.) are located on the roof, above the reach of flood waters.

In developing the pavilion-like theme, the architect took advantage of the durability and symmetrical beauty of structural steel framing. The superstructure is formed from lightweight structural steel framing. The two towers on either side of the stage house and a peaked structure over the stage house extend 126 ft higher than the stage floor level. These are constructed from a light, open steel framework, to create a transparent, web-like appearance. The towers on each side of the stage house are 33 ft square. The faces are framed using 8-in. wide-flange verticals and horizontals, spaced 11 ft each way. Each panel was X-braced with 4-in. WT's.

A box truss, 11-ft high by 11-ft wide, spans the 44-ft space between the tower and the stage house. Constructed entirely of eight wide-flange members, it serves to support the roof trusses covering the seating area. The rear of the seating area roof is supported on eight towers, 11-ft square by 22-ft tall, bridged by a box truss system. The tower vertical members are 6-in. wide-flange columns at each corner braced on a 5 ft-6 in. grid. The box trusses, the faces of the large towers and the small towers were fabricated in the shop to the largest shippable elements (11 ft by 11 ft by 44 ft) to save construction time in erection and assembly.

Design of the structural steel involved the use of computer techniques to analyze the framing for stresses and to minimize lateral deflections which result when very light sections are used. This approach also permitted control of secondary stresses that result from wind and thermal effects. The computer also permitted the designer to optimize the distribution of structural members, which minimized deflections while satisfying the stress conditions. Computer design of the structural and architectural steel resulted in a savings of almost $300,000.

Extending toward the rear behind the covered seats, the grade slopes gently upward to afford a good view of the performance for patrons seated on the grassy areas. The rear of this area is defined by a cast-in-place curved pergola, 750 ft long, which provides space for restrooms, concessions and ticket offices.

Foundations for the stage house and covered seating area are supported on 50-ton auger-cast piles, extending 40 ft into dense sand since the site has been covered with river silt and miscellaneous fill. The pergola is further back from the river, and effectively only one story high. It was decided to preload the bearing area with 4 ft of soil for three months before starting the construction of this structure. Taking advantage of pre-consolidation of the soil, together with keeping the soil pressure less than one and a half tons per square foot, minimized settlement to tolerable levels.

According to Architect Michael Graves, "The design attempts a fresh approach to the assembly of many people under one roof. In its simplicity, the scheme summons thoughts of a congregation under a tent, a building by the river and the relaxed atmosphere of a pavilion in the park." □

Open steel framework created web-like appearance for outdoor concert arena. Box truss between tower and playhouse supports trusses that cover seating area.

Architects
Michael Graves
Princeton, New Jersey
Carl A. Strauss & Associates (assoc. architect)
Cincinnati, Ohio

Structural Engineer
DeSimone, Chaplin and Associates
New York, New York

General Contractor
F. Messer & Sons Construction Co.
Cincinnati, Ohio

Steel Fabricator
Holston Steel Structures
Bristol, Tennessee

Owner
Cincinnati Symphony
Cincinnati, Ohio

Rohm and Haas Modifiers Plant: Modular Steel Construction Speeds Schedule

by Keith Flemingloss

Keith Flemingloss, P.E., is the Civil Engineering Specialist for Rohm and Haas Company's Central Engineering Division, Bristol, Pennsylvania.

Rohm and Haas Company confronted a major scheduling challenge head-on when, in August 1983, approval was given to begin detailed engineering on a major PVC Modifiers plant expansion for their Louisville, Kentucky facility. Marketing projections required the project be operational by the end of June 1984!

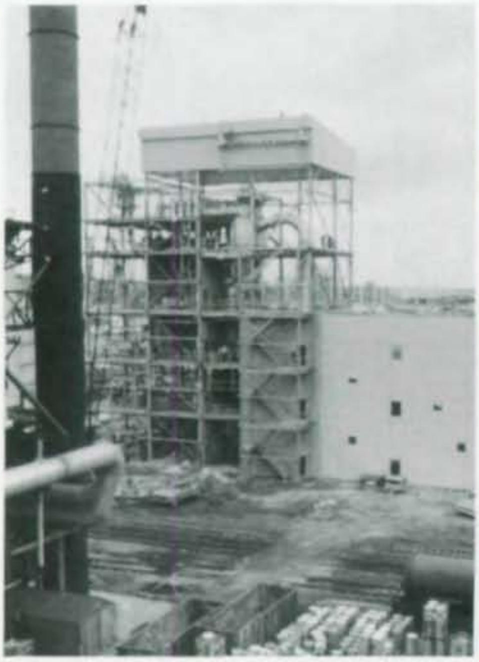
The chemical process facility consisted of a process building enclosing 34,150 sq ft, divided into many areas with uneven floor heights and supporting process equipment of every description throughout the structure. An enclosed tank farm building was required, with 3,240 sq ft weatherproofed for raw material storage. The product packaging building, with 6,800 square feet of floor space, completed the accelerated construction schedule. Together, these buildings required nearly 500 tons of galvanized structural steel framing.

Although time alone would have been a major concern, the schedule also demanded the process building be erected during January, February and March. While Louisville is not in an area noted for heavy snowfall, the winter weather in the area is noted for its variability, freezing rains and gusty winds. National Weather Service statistics showed an average of one in three days per week were likely to be lost for steel erection during this period.

After considering the hazards of winter erection, plus the anticipated downtime due to weather, a scheme of modular steel construction was devised. The complex main process building would be assembled at grade, one floor at a time, and stacked like building blocks to form the completed 108-ft tall building.

Design Phase

Since equipment layouts had been well developed, an immediate start on the actual analysis and design was possible in



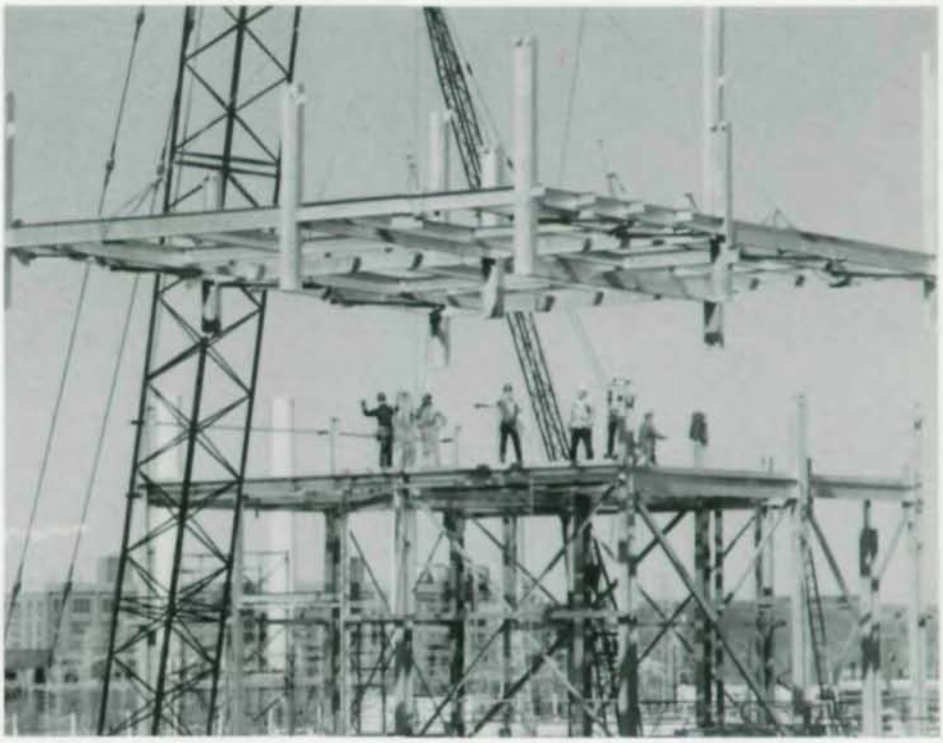
early September. The basic floor framing design was performed with Rohm and Haas' FLBEAM program. FLBEAM is given the basic floor layout, desired loadings, floor openings and any other pertinent data, and returns a plot of the floor framing, with all beams sized as simple beams of minimum section weight. The speed of this floor analysis allowed experimentation with various framing arrangements to optimize the steel framing requirements. An estimated 10-15% weight savings was realized in this case. Column loads are tabulated automatically as well, which eased the design of the columns and eliminated the waste of "worst case" design.

The modular erection scheme placed very few restrictions upon design of the framing. The equipment arrangement dictated a bay size layout that varied in both directions, with bay widths varying between 17 ft and 23 ft, and bay lengths between 20 ft and 25 ft. Unsymmetrical bay sizes and unsymmetrical framing due to equipment openings complicated the requirement that each module be capable of maintaining its shape while being lifted into place. To handle this requirement, semi-rigid connections were used on the interior girder-column connections. The actual modules were two bays square, with dimensions of 45 ft x 40 ft and 40 ft x 40 ft, and weighed between 16 and 21 tons each. The second module of each floor level framed into columns, which were erected as part of the preceding module. This was accomplished by providing erection seats to land the girders for alignment and bolt-up. The original design called for two splices each for the 15 columns involved in the modular part of the building. To allow each module to be assembled at a safe, convenient height from grade, extra column splices were required, and were designed five feet below top of steel for each level. To ease alignment during erection, a column splice detail was developed which combined erection clips similar to the type used in welded splice details, with bolted cover plates attached after the nine-column module was landed.

The fabricator's engineering group suggested another area which they felt could result in further savings—using shear plate connections in place of the traditional clip angles. After reviewing the AISC *Engineering Journal* article on the current state of the art in single-plate framing connections, this approach was accepted.

From the first transmittal of design drawings on Nov. 1st, 1983 it was apparent the schedule was going to be difficult to maintain. So, a decision was made to detail and fabricate the building as a sequence of modules, breaking the project into manageable units for expediting and monitoring progress. An added benefit resulted from incorporating design modifications to

Main scheduling challenge on Rohm and Haas Modifiers Plant (l.) was met with structural steel. Plant is shown before and after cladding.



upper floors during the original detailing, rather than as corrections or revisions in the field.

"Building Blocks" Go Up

The modular erection scheme required little in the way of extra construction facilities. The site had barely enough room for a module assembly area to the side of the building. Small, temporary footings were installed, and the modules assembled 58 ft south of their final destination.

The detailing and fabrication of one module at a time resulted in a sequenced delivery of materials which eliminated the requirement for a large "shake-out" or storage area, and permitted the completion of each module once it was begun.

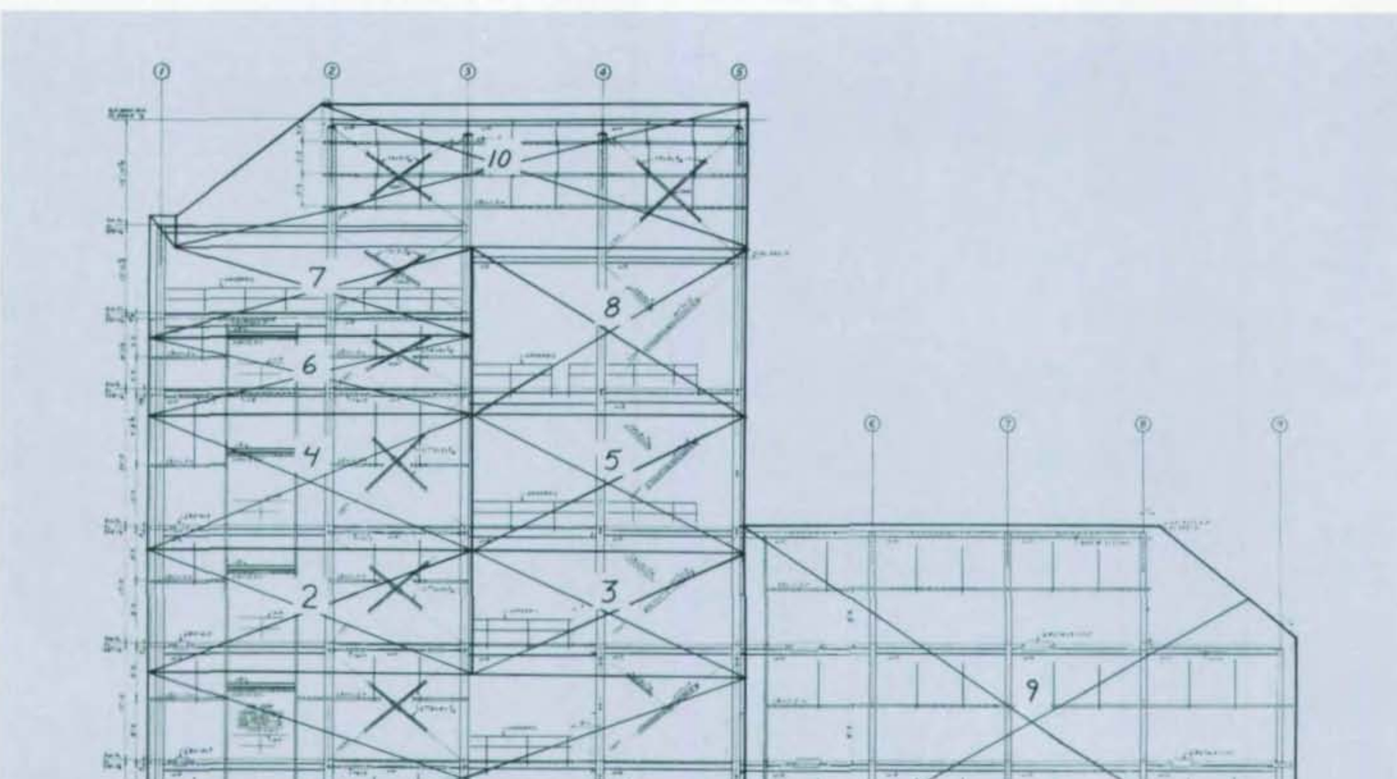
Module #1, the entire first floor of the main building, was erected in conventional fashion. The second and third modules were each half of the second floor, and were assembled side by side at grade. This was accomplished during a period of freezing temperatures and high winds. With a break in the weather, each module was lifted into place by a single crane. The lift and alignment of the nine column splices took less than three hours per module. This procedure was repeated through the next six modules, until the building had reached the 90 ft level (see sequence in Fig. 1). The roof framing was left off to permit equipment placement, and erected later in conventional fashion.

As the project progressed, other benefits became apparent. Due to the schedule pressure, it was important to enable as much productive work as possible, as quickly as possible. The modular scheme permitted an erection crew to assemble modules, while a detail crew installed bracing, handrails and flooring. This alone resulted in an apparent schedule savings of almost half the erection schedule for the main building. A subtle savings became apparent as the erection productivity was analyzed: the single-plate framing connections eliminated the need for erection bolting and allowed each bay to be completely assembled and bolted-up, independently of any surrounding bay. This meant that virtually all bolting could be done at grade, conveniently and safely.

Time *and* Cost Savings

On this project, it was always kept in mind that whatever construction method was used, whatever details were chosen, must help the project schedule, and still guarantee a safe, practical structure. For this project, the modular erection scheme resulted in a tangible savings in the schedule. Equipment which was originally planned to be stored at the site was erected directly into the structure by March—while the original construction schedule had shown steel erection *beginning* in mid-March! In fact, steel erection

Detail of steel module layout. Numbers indicate erection sequence.



began in mid-December and progressed continuously through the winter.

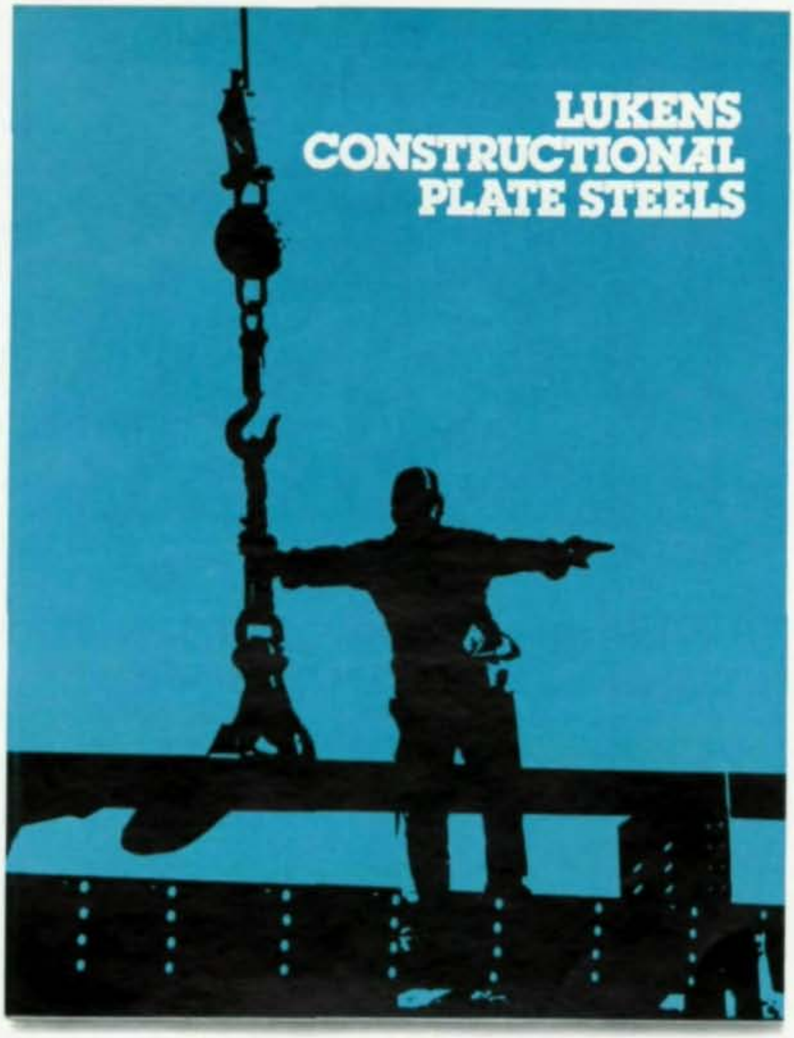
While the modular scheme saved the schedule, that was not its only influence. There were actually cost savings due to increased productivity, as well. Estimates near completion of the modular assembly and erection phase of the project indicated a savings of almost 30% of the original labor estimate. In addition to the fact that much of the assembly was done at grade, some of the increased productivity was due to the fact the ironworkers were enthusiastic about showing their ability to adapt to the new erection scheme. They offered many suggestions which simplified the erection of the modules.

Credit needs to be given, as well, to the fact there were virtually no detailing errors on the steel for the main building, and very few minor fabrication errors. □

Engineer/Architect/General Contractor
Rohm and Haas Co., Central Engineering Div.
Bristol, Pennsylvania

Steel Fabricator
Southern Ohio Fabricators, Inc.
Cincinnati, Ohio

General Contractor/Owner
Rohm & Haas, Kentucky
Louisville, Kentucky



When it comes to constructional plate steels, we wrote the book. This current edition features Lukens' capabilities with regard to:

Sizes. Standard specification plates available in widths to 195," lengths to 1250" and thicknesses to 25." A size card shows details.

Specifications. Mechanical properties and chemistry of the various grades of steel most frequently found in bridges and buildings. Displayed in chart form.

Heat Treating. Offered on plates up to 890" long.

Stripped Plate. An alternative to universal mill plate in applications such as fabricated bridge girders. Produced in lengths from 120" to 1250," widths 12" to 48" and thicknesses ¾" to 12."

Lukens-Conshohocken. A rolling mill and shipping complex designed to meet your needs for light-to-medium thickness carbon plate and our Sure-Foot® safety floor plate.

Lukens Fineline.® A family of low-sulfur constructional steels particularly effective when used in fracture critical applications.

For your copy of this brochure, illustrated with photos of our facilities and our products in use, just fill out the coupon below.

LUKENS STEEL

Write right now.

LUKENS STEEL COMPANY
586 Services Building
Coatesville, PA 19320

Please send me a copy of your brochure, LUKENS CONSTRUCTIONAL PLATE STEELS.

NAME ______________________

TITLE ______________________

COMPANY ______________________

ADDRESS ______________________

CITY ____________ STATE ____________ ZIP ____________

Throgs Neck Bridge: How Steel was Switched to American Suppliers

Any state, locality or company which produces construction materials or products for purchase by government agencies . . . in competition with overseas suppliers . . . would do well to study, and emulate, this example of switching an accepted, lower, foreign-supplier bid to that of American companies.

What could become the classic case of a program to successfully accomplish such a conversion involves a New York City bridge, a domestic steelmaker, fabricators and an erector.

The Throgs Neck Bridge, opened in 1961, is a 13,410-ft, six-lane suspension bridge linking the Bronx and Queens, two New York City boroughs. Because of deterioration in cantilevered deck approaches, its operator—the Triborough Bridge and Tunnel Authority, a division of the Metropolitan Transportation Authority (MTA)—determined that redecking parts of the approaches was necessary.

The project requires 16,000 tons of one in.-thick, A36 grade steel plate, with welded orthotropic ribs, fabricated in sections. New steel decking and an asphalt surface are to be fabricated in the shop, and then installed on-site by fastening and welding. Three lanes are being redecked in Summer/Fall of 1984; the other three in 1985. Sections of the bridge will be closed at night so that removal of the old deck and installation of the new minimizes interference with traffic.

Six bids were received in June, 1983 for the redecking project, including steel, fabrication and installation. The low bid, $32.9 million, accepted in July from Karl Koch Erectors, of Carteret, N.J., was based on steel to be produced in Japan and fabricated in South Korea. About $13 million of the total bid was for steel and its fabrication. It had been determined that using American steel and fabrication resulted in a bid $3.4 million higher.

What initiated intense opposition to the foreign portion of the bid was the fact that, at the same time, a "Buy-American Steel" Act became law in New York State. Under its provisions, state agency purchases must be of domestically produced steel, unless the public contracting authority could prove that using American steel results in "unreasonable cost," not further defined in terms of figures or percentages.

A major American steel producer, who fabricated and erected 10,000 tons of steel when the Throgs Neck Bridge was first built, did not believe that $3.4 million constituted "unreasonable cost," and helped orchestrate a full program to have the foreign portion of the contract rescinded.

The sole argument in favor of the accepted bid was the $3.4-million differential between foreign and American steel. It could help subsidize fares on New York City's bus and subway system as well as suburban commuter rail systems, or it could be used to help fund additional capital projects.

Marshalled against that argument were economic and other justifications for steelmaking and shop construction in the U.S. With regard to the crucial issue of jobs, steel plate for redecking would be produced in Maryland at an approximate rate of one ton per steelworker per day (8-hr. shift). Thus, the total tonnage would provide 16,000 man-days of employment at a wage-and-benefits pay scale of $168 per day, for a total of $2,688,000 in pay and $806,400 in taxes (at a 30% rate).

The 4,028 tons of orthotropic rib sections to stiffen the bridge's steel plate deck would be manufactured in a New York steel specialty shop and require 1,560 man-days. Estimates are that this would, in turn, support an additional 3,120 man-days of indirect work in the Niagara area. Fabrication of the bridge deck panels is a

Throgs Neck Bridge, New York City, undergoes huge redecking program during 1984-85.

joint venture of two specialty shops. The final result of their work is fully completed panels, about 19-ft wide by 50-ft long, including all of the steel components and a sub-base asphalt surface ¾-in. thick. Work at the two companies, involving 80 men each, totals 40,000 to 50,000 man-days per year.

Although tax calculations are not yet available for the Throgs Neck Bridge contract, this analogy may suffice. Studies have shown that, when the U.S. Government purchases $1-million worth of defense materials from a domestic supplier, the money moves about the economy with a feedback effect. Through company taxes, payrolls, dividends and other purchases, $524,000 returns in taxes. Instead of a domestic purchase costing the government $1 million, it actually costs only $476,000 or 47% of the initial expenditure.

Other benefits include a heightened safety factor, for in the event a faulty section is produced by the New York fabricators, it is almost immediately replaceable. Contrast that with an 11,000-mile supply line from foreign sources! In practical terms, how does an inspector reject a section not up to specifications if he knows its closest replacement is in South Korea?

Backing up all this economic justification was intense lobbying and publicity at both state and local levels. Lobbyists visited, called and wrote key New York State legislators, senators and assemblymen, pointing out the inconsistency between the MTA's action and the newly enacted Buy-American Steel law. New York State's Commerce Commissioner, William Donahue, was contacted and he promised his full cooperation.

The American Iron and Steel Institute, which has a Buy-American task force, communicated with Governor Mario Cuomo, and made widely known New York's stake in the steel industry's economic recovery. For example, two major American steel companies alone purchase annually more than $1 billion in New York goods and services.

A Bronx assemblyman set up a news conference on the steps of MTA's office in New York City. Editorial support for reversing the decision and buying American steel was received from newspapers throughout the state, although some downstate papers were in opposition. Key newspapers in Buffalo and Albany were contacted, and it was called to their attention the decision to buy foreign. Top officials of the New York State AFL-CIO and the United Steelworkers of America became actively involved.

Governor Cuomo was the key to MTA's reversal of its decision. Late in September—after the Governor asked it to reconsider the Throgs Neck contract—the MTA board changed its mind and voted to require the use of domestic steel plate fabricated in New York.

Thus, a bid encompassing foreign steel and fabricated parts was switched to American steel and parts. In addition, at least 66,000 man-days of employment will be achieved for U.S. workers, and more than the $3.4-million differential will be returned in taxes. How this changeover was accomplished should be especially important to those in the majority of states which already have Buy-American laws. It was the presence of such a law in New York State which provided a statutory base for all arguments and helped immeasurably in overturning the foreign bids. □

We are indebted to the Steel Products News Bureau of AISI for this information and photo.

The Dulany Condominiums: Mother Nature Takes a Backseat!

The Dulany Condominiums, at the base *of Mt. Werner, Steamboat Springs,* Colo., boasts an intricate five-level structure that reflects the image of the bustling Colorado ski resort. And yet it blends with the surrounding rustic atmosphere of the Colorado community.

Flexibility of architectural design and speed of erection are the focal reasons why structural steel framing was selected for the 25-unit project. According to Stephen J. Campbell, president of the structural engineering firm, "The client wanted a flexible structural system which would permit the desired aesthetics and form required for the project, while maintaining the desired economy. In addition, the fast-track construction approach to the project, necessitated by the short construction season in the high country of Colorado, played an influential role in the selection of the final framing system. The selection of structural steel framing solved inherent design and construction problems as well as lending considerable aesthetic flexibility."

Preliminary Framing Analysis

Once the basic condominium unit plan (Fig. 1) was established in relation to the overall layout, several structural systems were considered: precast concrete hollow-core slabs supported on steel frame; precast concrete hollow-core slabs supported on precast concrete walls and *beams; and a structural steel frame with a* composite floor system.

Experience with several similar projects led the structural engineer and the contractor to conclude a structural steel frame with a composite floor system and a beam and steel joist roof would provide optimum flexibility and economy while achieving swift erection.

Fast-Track Beats Mother Nature

The project engineer, A. J. Baysek, explained, "Structural steel was selected for several reasons, the most obvious being versatility. Structural steel is ideally suited for fast-track construction. In addition, it provided the architect with a versatile and flexible structural system to assist in the adaptation of the building form to the project's physical demands, and it helped reduce the overall height of the building.

"It also provided the lightest framing system, which minimized the vertical loads to the foundation and contributed to overall cost savings. We were able to use spread footings instead of a more costly foundation system, such as piles. Even then, the lighter dead load minimized spread footing sizes, important because of the site's low soil-bearing capacity. Structural steel framing offered many advantages for the criteria established for the project; speedy erection . . . design flexibility . . . economy . . . and architectural and structural *alterations could be made simpler once the job* was in progress."

Campbell also pointed out, "Since we were forced by Mother Nature into a fast-track approach to the project, using structural steel allowed us to reduce construction time considerably. The structural steel mill order was placed before the drawings were completed. Foundation construction then began as early as possible and was in place when the fabricated steel arrived."

Floor Construction Easier

The floor system (Fig. 1), spanning approximately 14 ft between beams, consists of 3-in. composite steel floor deck topped with 3¼-in. lightweight concrete with composite steel beams (composite action achieved by welding 3/4-in. dia. studs through the deck to the beams). This system enabled the structural engineer to provide a light, economical framing system which minimized overall structural depth.

Each condominium unit was designed with individual cantilevered balconies. The balconies were designed for unusually high live-load requirements, because of snow accumulation and the storage of firewood for the long winter months. The adaptability of structural steel framing to the detailing of such cantilevered construction enabled the designers to achieve the desired function and aesthetics of the balconies within the construction depth limitation (Fig. 2).

The structure was designed to resist not only the 25 psf lateral loads induced by wind, but also the Zone 1 seismic loads required by the Uniform Building Code. Lateral resistance is provided by K-braced bents which transfer the horizontal forces to the foundation. The K-braced bents were strategically placed in the demising walls between units.

Intricate Framing Made Simple

The roof profile was a combination of sloped, gabled and shed-type roofs,

Dulany Condominiums, Steamboat Springs, Colo.

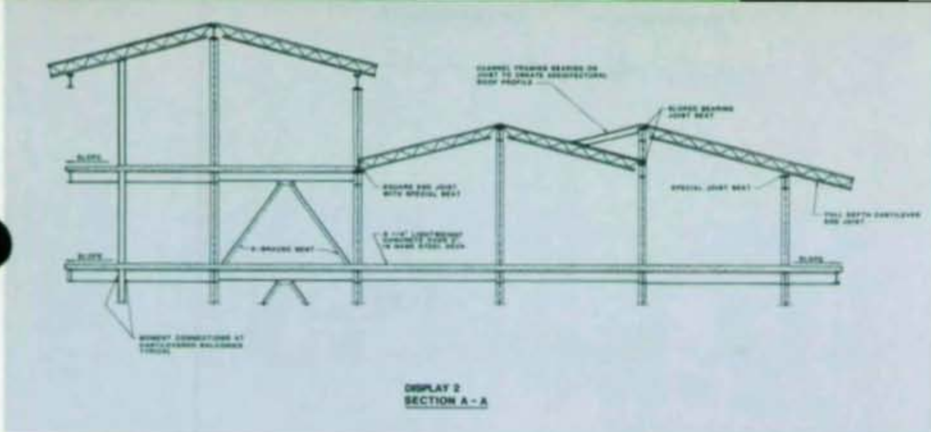


Figure 2

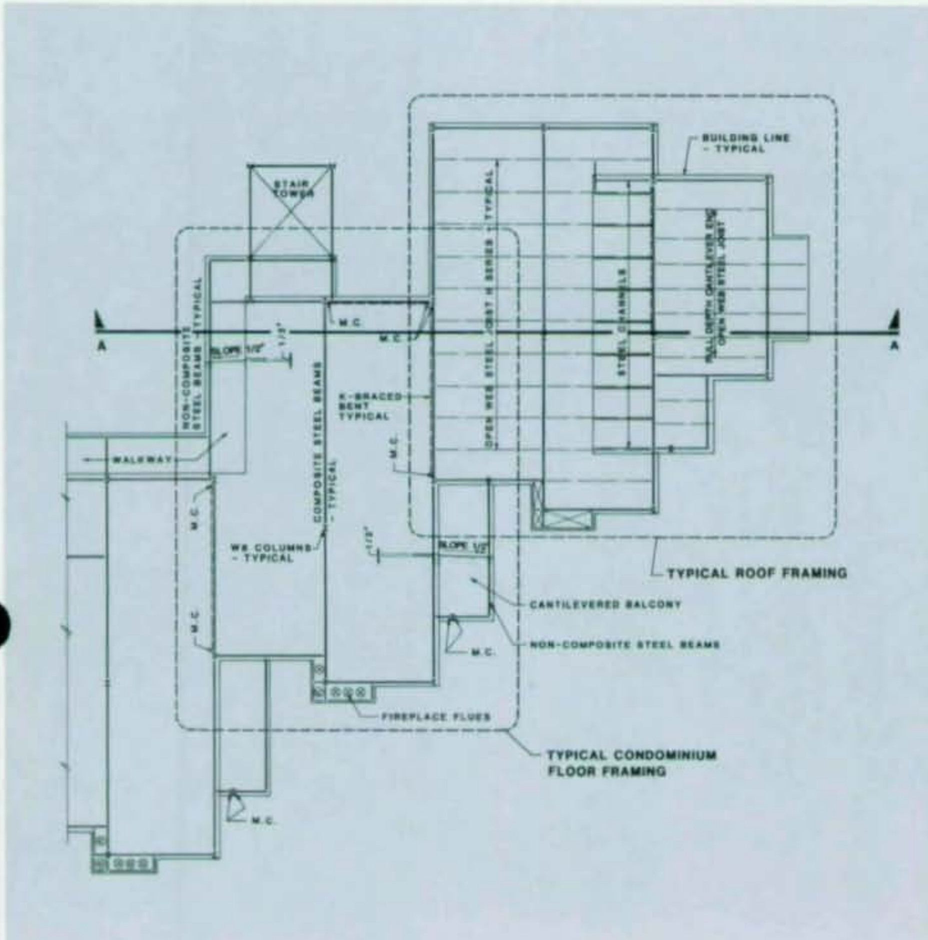


Figure 1. Partial framing plan (5th level)

which allowed the architect to achieve the desired interior ceiling elevations and slopes for the individual condominium units and yet provide an aesthetically pleasing overall roof profile. The use of steel beams and open-web steel joists permitted the versatility needed to accommodate the intricate variations of the design (Fig. 2).

Campbell, in summing up the project, commented, "The selection of structural steel allowed us to maintain an economical unit without having to compromise the architectural expression." □

Architect
Dallas Taylor/Thomas E. Woodward & Associates, Inc.
Dallas, Texas

Structural Engineer
Gunnin-Campbell Consulting Engineers, Inc.
Dallas, Texas

Contractor
Hensel-Phelps Construction Co.
Greeley, Colorado

Owner
Dulany Corporation
Steamboat Springs, Colorado

Structural steel framework shows tiering effect of gable roofs (top). K-braced bents are used for lateral resistance (c.). Cantilevered balconies, with typical floor framing (bott.).

AMERICAN INSTITUTE OF STEEL CONSTRUCTION
The Wrigley Building, 400 North Michigan Avenue
Chicago, IL 60611

Address Correction Requested

BULK RATE
U.S. POSTAGE
PAID
WHEATON, ILL.
Permit No. 1786