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# MODERN STEEL CONSTRUCTION

NUMBER 6 • 1988

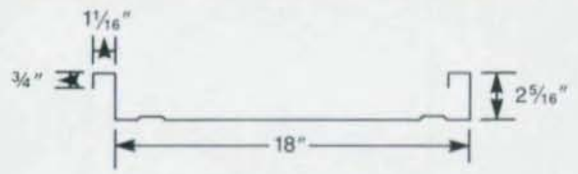
AMERICAN INSTITUTE OF STEEL CONSTRUCTION INC. AISC FOUNDED 1921

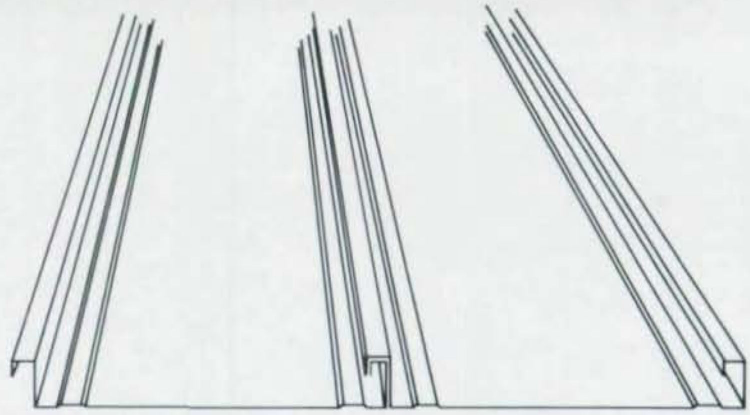
## THIS ISSUE

# USD UNITED STEEL DECK, INC.

P.O. BOX 662, SUMMIT, NJ 07901 (201) 277-1617

# SS18 STANDING SEAM ROOF PANELS



**OPTIONS AVAILABLE**

- MATERIAL – GALVALUME
  - ALUMINIZED STEEL
  - G90 GALV.
  - G90/PAINTED
- PAINT – SILICONE POLYESTER
  - KYNAR
- LENGTH – 6′ TO 60′

## ALLOWABLE UNIFORM LOADS, psf

| Deck Description | Span ft. | One Span | | | Two Spans | | | Three Spans | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | Roof Stress | Wind Stress | L/240 Defl. | Roof Stress | Wind Stress | L/240 Defl. | Roof Stress | Wind Stress | L/240 Defl. |
| | 4.0 | 123 | 151 | 189 | 113 | 151 | 563 | 142 | 189 | 441 |
| 24 gage; t = 0.0238 | 5.0 | 78 | 97 | 97 | 73 | 97 | 288 | 91 | 121 | 226 |
| I positive = 0.272 | 6.0 | 54 | 67 | 56 | 50 | 67 | 167 | 63 | 84 | 131 |
| S positive = 0.147 | 7.0 | 40 | 49 | 35 | 37 | 49 | 105 | 46 | 62 | 82 |
| I negative = 0.184 | 8.0 | 31 | 38 | 24 | 28 | 38 | 70 | 35 | 47 | 55 |
| S negative = 0.136 | 9.0 | 24 | 30 | 17 | 22 | 30 | 49 | 28 | 37 | 39 |
| | 10.0 | 20 | 24 | 12 | 18 | 24 | 36 | 23 | 30 | 28 |
| | 5.0 | 101 | 119 | 119 | 90 | 119 | 362 | 112 | 149 | 284 |
| 22 gage; t = 0.0295 | 6.0 | 70 | 83 | 69 | 62 | 83 | 210 | 78 | 104 | 164 |
| I positive = 0.346 | 7.0 | 51 | 61 | 43 | 46 | 61 | 132 | 57 | 76 | 103 |
| S positive = 0.189 | 8.0 | 39 | 47 | 29 | 35 | 47 | 88 | 44 | 58 | 69 |
| I negative = 0.227 | 9.0 | 31 | 37 | 20 | 28 | 37 | 62 | 35 | 46 | 49 |
| S negative = 0.168 | 10.0 | 25 | 30 | 15 | 22 | 30 | 45 | 28 | 37 | 35 |
| | 11.0 | 21 | 25 | 11 | 19 | 25 | 34 | 23 | 31 | 27 |
| | 6.0 | 86 | 100 | 83 | 75 | 100 | 255 | 94 | 125 | 200 |
| 20 gage; t = 0.0358 | 7.0 | 63 | 73 | 52 | 55 | 73 | 161 | 69 | 92 | 126 |
| I positive = 0.423 | 8.0 | 48 | 56 | 35 | 42 | 56 | 108 | 53 | 70 | 84 |
| S positive = 0.231 | 9.0 | 38 | 44 | 25 | 33 | 44 | 76 | 42 | 55 | 59 |
| I negative = 0.274 | 10.0 | 31 | 36 | 18 | 27 | 36 | 55 | 34 | 45 | 43 |
| S negative = 0.203 | 11.0 | 25 | 30 | 14 | 22 | 30 | 41 | 28 | 37 | 32 |
| | 12.0 | 21 | 25 | 10 | 19 | 25 | 32 | 23 | 31 | 25 |
| | 7.0 | 82 | 86 | 62 | 65 | 86 | 202 | 81 | 108 | 158 |
| 18 gage; t = 0.0474 | 8.0 | 63 | 66 | 41 | 50 | 66 | 136 | 62 | 83 | 106 |
| I positive = 0.556 | 9.0 | 50 | 52 | 29 | 39 | 52 | 95 | 49 | 65 | 74 |
| S positive = 0.303 | 10.0 | 40 | 42 | 21 | 32 | 42 | 69 | 40 | 53 | 54 |
| I negative = 0.322 | 11.0 | 33 | 35 | 16 | 26 | 35 | 52 | 33 | 44 | 41 |
| S negative = 0.238 | 12.0 | 28 | 29 | 12 | 22 | 29 | 40 | 28 | 37 | 31 |
| | 13.0 | 24 | 25 | 10 | 19 | 25 | 32 | 23 | 31 | 25 |

## MAXIMUM UNPLANKED SPANS FOR CONSTRUCTION AND MAINTENANCE LOADS

| Gage | 24 | 22 | 20 | 18 |
|---|---|---|---|---|
| Single Span | 4′-9″ | 6′-0″ | 7′-0″ | 9′-6″ |
| Multi Span | 6′-0″ | 7′-6″ | 9′-0″ | 12′-0″ |

Avoid loading panel ends until they are fastened.

Roof loads are based on a limiting stress of 20000 psi; wind stresses are increased by 1/3. Wind and deflection loads consider either pressure or suction – the least value is shown.

# Structural Software Company

## Material Allocation Programs




### If you want to drive a shop, as well as a plotter

with your **IBM-AT** or compatible computer then you need the Structural Software Company family of programs. The information you input to detail a job, using our **FabriCAD** system, can also update the other members of the family. The **SSC Estimating** program can take the information the Detailing program supplies, and craft it into reliable forecasts of labor and material costs.

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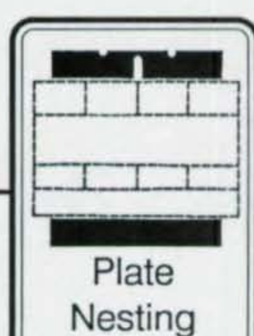
From beginning to end, and at every step in between, the SSC family of programs improves your control, reduces your expenses, and increases your profit margin.




For more information on these programs, and others, write to us at PO Box 19220, Roanoke, VA, 24019. Or call us at (703) 362-9118.

# MODERN STEEL CONSTRUCTION

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**American Institute of Steel Construction, Inc.**

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**OOPs Dept.!**
In Issue 4, on the GMF Robotics feature, we neglected to credit the fine architectural photos of Beth-Singer, Franklin, Michigan. Our apologies.—Ed.











# WOODY HAYES CENTER

## A Case for Flexibility

by David A. Holtzapple

Every university strives to provide the best of facilities for its students. The aim from the very outset on the design of the new Woody Hayes Athletic Center for Ohio State University was just that. This new multi-sport facility fulfills the indoor practice needs of a whole variety of sports, ranging from football and baseball to soccer and field hockey. In addition to providing a large enclosed unobstructed area for practice, the center also provides support facilities in locker, meeting and weight training rooms. Throughout the facility, efforts were made to equal, if not exceed, the best features of similar facilities throughout the country. The result is a premier athletic facility of which The Ohio State University can be truly proud.

The Woody Hayes Athletic Center is actually a complex of buildings and fields related to athletics. Located just northwest of the main university campus, it has three major building elements: a 90,000 sq. ft indoor practice field, a new 40,000-sq. ft support facility and the existing 30,000-sq. ft Biggs Building.

The indoor practice facility provides a 220-ft wide by 400-ft long practice field with an artificial playing surface suitable for a wide range of sports. It is also equipped with suspended netting to allow the field to be subdivided temporarily so it can be used by more than one sport at a time.

The new 40,000-sq. ft support facility contains an 8,000-sq. ft weight training room, a 4,000-sq. ft locker room, 5,200-sq. ft of team meeting rooms and conference areas and a 3,000-sq. ft main entrance lobby which connects new facilities with existing ones, as well as provides a dramatic public space to showcase the State University athletic achievements. The existing Biggs Building, completely renovated as part of this new project, provides locker, training and laundry facilities for both men and women athletic teams. Also part of this complex is a new, natural grass outdoor practice field just south of the indoor field.

### Extensive Tours on Design

Previous to design of the new facility, an extensive tour was conducted of similar

*Ohio State's new Woody Hayes indoor athletic center, Columbus, O.*

*Public entrance to new facility connects new to existing areas, provides dramatic trophy showcase for athletic awards.*

*Horizontal truss element of typical bent, supported by temporary erection frames.*

ones throughout the country. The basic purpose was to find out what problems others had experienced, observe the solutions, and from this choose the best alternatives for the new complex. Operational and functional, as well as physical, aspects of these facilities were studied and evaluated. From the engineering point of view, these included both major systems—structural framing, heating, ventilating and lighting—and individual elements, down to such details as faucet arrangement in shower rooms. Many of the design objectives for the center were established and refined as a result of these visits to other facilities.

Some of the major design parameters for the indoor practice field included setting the clear height at a 65-ft minimum, requiring inside surfaces be smooth with no projecting columns which might lead to injury and establishing a quiet, effective ventilation system. The character and functions of the support area were also determined from the various visits to the other sites. It was desirable to have uncluttered and well-organized locker and training rooms, along with necessary weight-training facilities. Another important feature of the support facility was to provide an exciting display area for the many awards for Ohio State's long tradition of athletic excellence.

### Indoor Practice Facility

At the beginning of the design process many different types of long-span structures were studied. These, among others, included space frames and air supported structures. The final choice of rigid steel bents was made based on economy, speed of fabrication and erection and their ability to adapt easily into the architectural concept of the building. Steel provided the best overall solution to this unique problem.

The basic structure facility, rigid steel bents 36-ft o.c., spanning 220-ft across the field, were designed as tied, two-hinged frames. The columns on either side of the bent are five-ft deep, built-up plate members tied together with double-channel tie rods under the playing field. The horizontal element of the frame is a 10 ft deep steel truss rigidly connected to the columns at both top and bottom chords. The bents weigh about 71 tons each, most of which is high-strength (fy = 50 ksi) steel. Spanning the frames are conventional steel joists 6 ft o.c. A series of wide-flange girts span the main columns and support interior and exterior prefinished metal skins.

The lateral stability of the building was achieved in the narrow direction through the rigidity of the frames; in the long direction, vertical "X" braces were located in each long wall and horizontal wind trusses in the plane of the roof at each end. The use of a standing seam metal roof precluded using the roof as a diaphragm.

The large spans and high columns presented some rather special handling and erection problems. The 60-ft high, built-up plate columns were erected first, including attachment of double channel tie rods. The 220-ft long trusses were shop-fabricated and shipped in two parts to the site where, by special erection frames, they were spliced with full-penetration welds at the center. These frames then served as lifting devices for two cranes, operating in tandem, to lift the trusses. The cranes had to hold the trusses in place until both the top and bottom chords of the trusses were welded to the columns, and the diagonal "X" bridging installed. Only after all welds were completed would the frames become self-supporting, allowing the cranes to move on to the next bent. The details allowed for some adjustments that may become necessary due to accumulated fabrication tolerances and daily temperature variations. As a result of the relatively long spans and heights, there were no major problems with alignment and erection.

The requirement that there would be no projections into the field meant the 5-ft deep columns had to be flush with the interior of the wall. The architect used this to advantage in incorporating two other features. The deep wall cavity allowed for a stair and observation platform to be included without interruption in the regular architectural pattern. This platform can be used by coaches to observe practice from 50-ft above the field. In addition, it provides a window overlooking the outdoor practice field as well. Direct sun from windows is undesirable when practicing most sports, so the architect again used the deep wall cavity to provide a screen on the inner wall to provide for indirect natural light. Coupling this natural light with reflective white surfaces of the walls and roof provides an inexpensive lighting supplement to artificial floodlights.

### Support Facility

Attached directly to the indoor practice field is a one-story conventional steel structure to serve all auxiliary functions of a football team. A large, main locker room with 150 custom wood lockers are laid out so a coach can exit his office and have direct eye and voice contact with anyone in the room. A 140-seat team auditorium with remote controlled AV equipment has a large team room which can be subdivided quickly into eight squad rooms. The training room has custom-designed taping and treatment tables and two adjacent examining rooms for doctors. Whirlpools and a large walk-in therapeutic pool are also access directly from the locker room.

For students and visitors alike, the new Woody Hayes Athletic Center is an inspiring example of Ohio State's commitment to excellence as well as an example of the versatility of structural steel to adapt to the needs of a unique structure. □

**Architect**
Patrick & Associates

**Associate Architect/Civil Engineer**
Moody/Nolan, Ltd.

**Structural Engineer**
Korda/Nemeth Engineering, Inc.
(structural, mechanical, electrical)

**General Contractor**
Peterson Construction Company

**Steel Fabricator**
The J. T. Edwards Company

**Owner**
The Ohio State University

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*David A. Holtzapple, P.E., is an associate with Korda/Nemeth Engineering, Inc., Columbus, Ohio.*

# NORTHERN STATES

## Designing for Power—with Steel

by Yan Shagalov and Sami Karam

The Northern States Power Resource Center is a $14.2-million, 280,000 sq. ft industrial and central material supply facility. Located in the northern Minneapolis suburb of Maple Grove, occupancy is scheduled for late 1988. Approximately 130,000 sq. ft of the center serves as a materials management center; 70,000 sq. ft houses a transformer repair shop and the remaining areas will be allocated to office and miscellaneous service spaces. NSP's intent was to construct a transformer repair shop containing maintenance and rebuilding capabilities for heavy electrical transformers weighing up to 110 tons, thus, exceeding the 25-ton capacity of their existing facility. Such maintenance tasks require the use of traveling overhead cranes of various sizes, spans and capacities. The presence of these heavy, moving cranes within the building was the most challenging issue for the structural engineer.

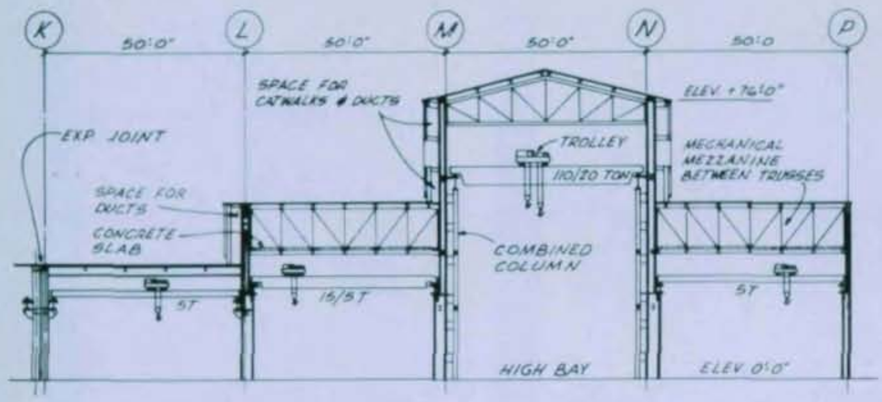
### Building Made for Steel

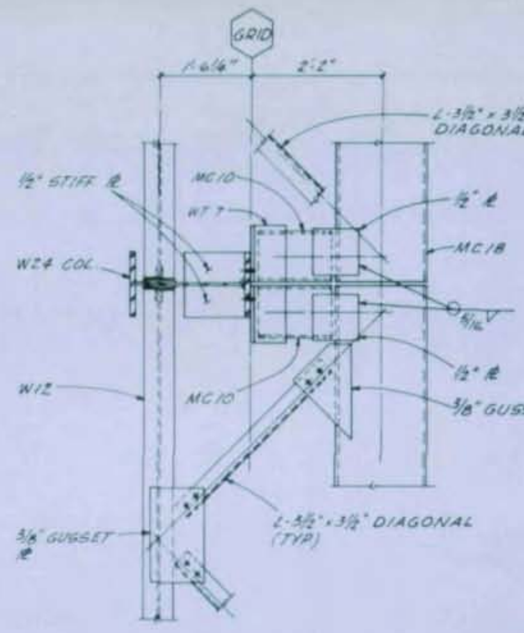
Early in the design process, steel was selected as the structural material. The advantages of steel were highlighted by concerns about the weight of the building, the impact and fatigue stresses created by the recurring crane loads and the required longevity of the structure, as well as economic parameters such as material delivery and construction time.

### Transformer Repair Shop

The structure accommodates 11 traveling overhead cranes with lifting capacities from two to 110 tons. Each crane girder was designed for the high gravity loads, as well as for the incidence of horizontal forces created by the bridge and trolley in both the transverse and longitudinal directions. These forces, largely due to the stop-and-go nature of the crane mechanisms, were considered as acting as impact loads. The design parameters of the crane areas resulted in the following:

- *High gravity wheel loads recurring in fatigue-inducing cycles.* A 25- × 50-ft bay was used for the crane facility; a 50- × 50-ft bay size was selected for the adjacent materials management center. The choice of a smaller span in the crane areas was prompted by the decision to use W rolled sections rather than built-up, crane-plate girders. Standard sections have the advantages of higher allowable stresses and a more favorable fatigue stress category. All crane girders were designed as simple spans to confine their moment envelope to the positive region and reduce the stress range to a manageable limit during each loading cycle. The absence of negative moments along each crane girder eliminated the need for lateral bracing of the bottom flange. Also, simple-span crane girders are unaffected by differential settlement of the supports. As recommended by AISE Technical Report No. 13, crane girder deflections un-



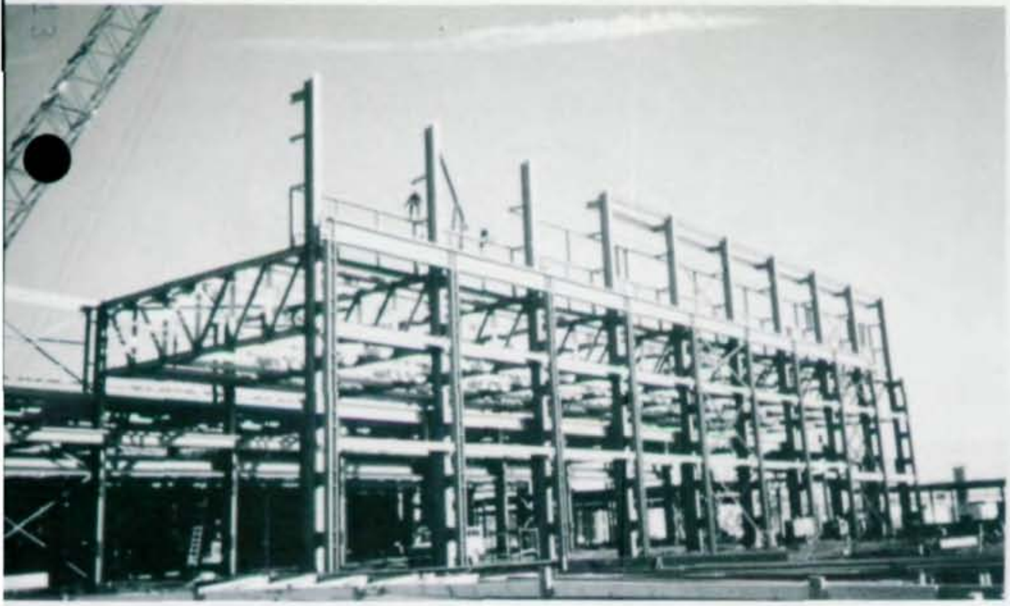



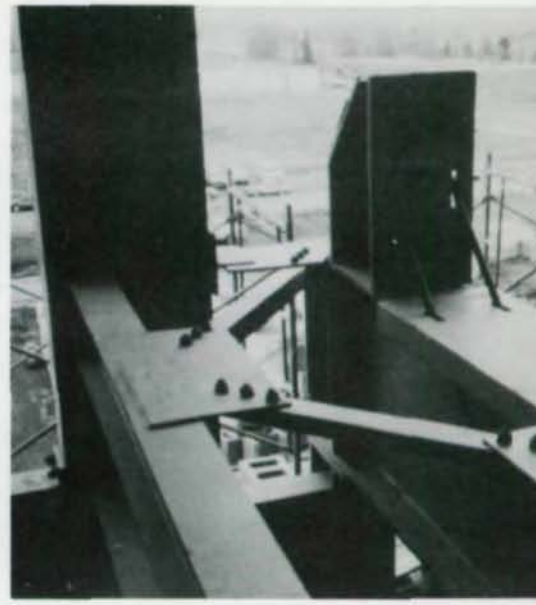

*Wind loads resisted by moment frame of four 50-ft bays (above). Detail of horizontal truss (r.). Transverse thrust resisted by horizontal truss.*

der live loads were designed to be less than 1/1000 of the span length.

- *Transverse crane loads acting at rail levels.* These horizontal forces acting near the top of the crane girders and perpendicular to the direction of the web required bracing of the top flange. For the smaller capacity cranes, the effect of these forces was counteracted by welding a horizontal channel to the top flange of the girder. In the case of the heavy cranes, the transverse thrust was resisted by a horizontal truss between the top flange of the crane girder and the adjacent primary structural frame.

- *Differential horizontal deflections between the two lines of rails for each crane.* Proper operation of each crane requires the two rails on each side of the traveling bridge will not displace away from each other in excess of one inch. This lateral deflection was found to be especially critical in the high bay area, where the hea-





viest crane (110-ton) is 45 ft from the floor. At that height, the horizontal displacements caused by the transverse loads from the crane are compounded by the incidence of wind deflections which, in some loading cases, separate the two rails further.

- *Combine columns.* In all cases but one, columns played a dual role as part of the primary building frame and as supports for the crane girders. However, in the high bay area, the magnitude of the loads was such that no single column standard wide-flange shape could resist simultaneously the effect of the wind on the frame (76 ft high) and carry loads of the 110-ton crane. To solve this problem, the crane bridge in this bay is supported by a combined column made up of two vertical column lines spaced 3 ft-2 in. c. to c. and attached by a series of short horizontal beams fixed at both ends. The combined column, therefore, acts as a vertical Vierendeel truss.
- *Crane runway stops.* Safety considerations require a crane stop be provided at the end of each rail line. The size and strength of each stop was determined by the size and capacity of its crane.
- *Primary moment frame.* Wind loads acting perpendicular to the direction of crane travel were resisted by a moment frame of four consecutive 50-ft bays of varying heights (30 to 76 ft). Because of large heights and widths of the bays, it was apparent from the outset that developing the necessary frame action would require deep trusses at the top of the structure. Each truss, fixed at both ends, helped reduce the lateral drift caused by the simultaneous action of wind and transverse crane forces. In the overall frame computer analysis, the trusses were modeled as single members using their composite sectional properties. This helped determine the necessary stiffness of each truss. A different computer model was subsequently created for each truss to specify the geometry and sizes of all the component members. The same two-step method was used for the computer analysis and design of the composite columns and for their contribution to the frame. All 50-ft long trusses and 76-ft tall composite columns were shop-built and delivered completely assembled. The space between the trusses on either side of the high bay was used as a mechanical mez-

zanine with a composite concrete floor spanning bottom chords of the trusses.

### Materials Management Center

Typical bay size in the materials management center is 50 × 50 ft. This wide spacing between the columns allows great flexibility for the lifting and movement of equipment. The concrete slab on grade is 4 to 12 in. thick and was designed to withstand uniform loads up to 2,000 psf or concentrated loads up to 30 kips spaced 8 ft o.c. A highly durable trap-rock floor finish was used on the slab during concrete placement. The roof, with a clear height of 28 ft is supported by square steel tube columns which are part of a series of moment frames in one direction. The choice of a tube, instead of a W section, makes each column equally strong in both axes, renders maintenance easier and reduces the risk of accidental bending of the flanges during the movement of heavy equipment. The roof structure is steel deck, steel joists and joist girders.

In addition to providing basic architectural/engineering services, the architect/engineer assisted NSP in these activities: programming, industrial engineering, site selection, evaluation of contamination protection for the site soils, liaison with local officials and community groups and selection of special consultants. Equipment selection and specifications, including traveling overhead cranes, were handled by Conkey/KWM, Inc. of Minneapolis. The entire design and construction process was characterized by a productive relationship between architect, engineer, client and contractor—an ideal condition for any project and a necessity in this case, because of the constraints of a short schedule. On this project, 1,667 tons of structural steel and 361 tons of joists were erected in 16 weeks by a construction team of 15. Fabrication and construction deadlines were met and the facility became operational in December 1988. □

**Architect/Engineer**
Hammel Green and Abrahamson, Inc.
Minneapolis, Minnesota

**General Contractor**
PCL Construction Services, Inc.
Minneapolis, Minnesota

**Steel Fabricator**
The Hustad Company
Minneapolis, Minnesota

**Owner**
Northern States Power Company
Minneapolis, Minnesota

*Yan Shagalov, P. E., is a senior associate in structural engineering; Sami Karam is a structural engineer with Hammel Green and Abrahamson, Inc., architects and engineers, Minneapolis, Minnesota.*

# UNITED AIRLINES HANGAR

## Fast Track *Beats* "On-Time" Arrival

by Ranjit Roy

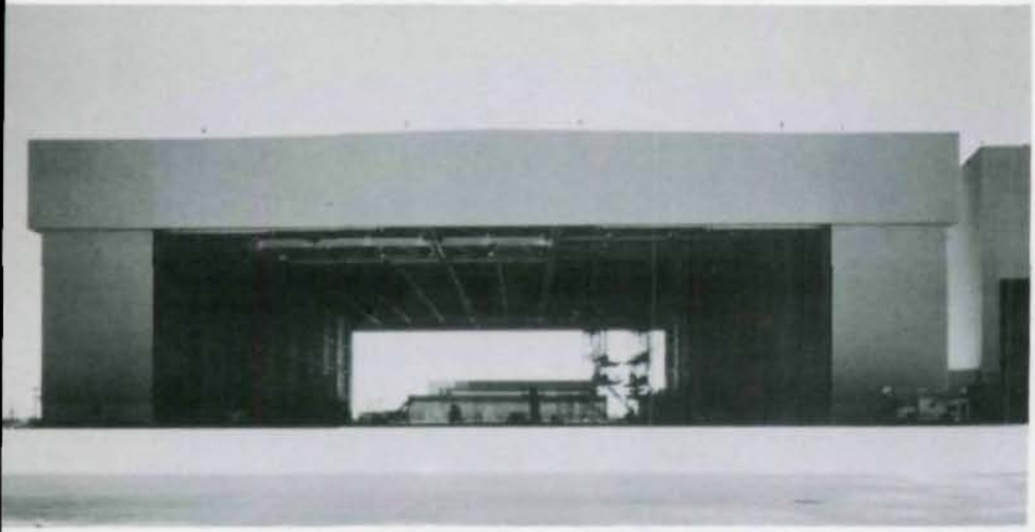
United Airlines has expanded its aircraft service center at Chicago's O'Hare Airport with a new 81,000 sq. ft hangar for the maintenance of their wide-body aircraft. In addition to the hangar, an adjacent core building of 20,000 sq. ft was constructed to connect the new hangar with the existing service center. The core building houses mechanical equipment, maintenance shops, classrooms, foremen work areas, parts storage as well as lunch and restrooms. This new state-of-the-art hangar was built in 10 months from the conceptual design to beneficial occupany by United.

The new hangar permits United to increase the maintenance check activity at its Chicago hub in response to expansion of its current fleet of wide-body aircraft. The hangar has the capacity to accommodate two DC-10s or two B767s simultaneously, or one B747. Urgency of the O'Hare hangar installation, vital to United's maintenance operation, led them to opt for a fast-track, turnkey project—a one-source responsibility for the total operation.

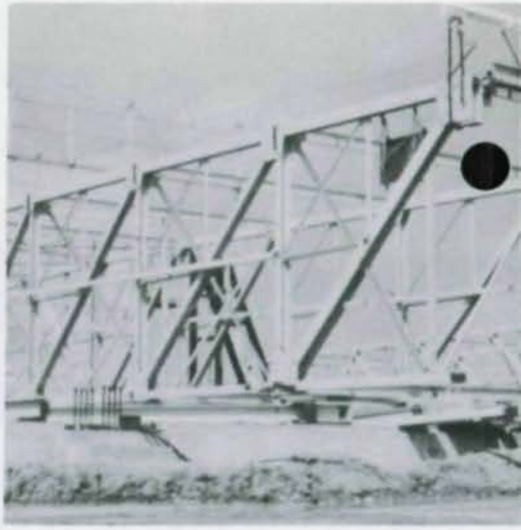
**Design Criteria**

Simplicity of structural framing and connection details were the overriding criteria of design to speed up the procurement, detailing, fabrication and erection of the hangar. The fabricator was selected by bidding on the preliminary design and the mill order of the jumbo steel sections proceeded simultaneously with the final design. By the time the final drawings were issued to the fabricator, the mill-ordered steel was ready at their shop.

The hangar has a plan area of 260 by



*Spacious interior of new United Airlines hanger accommodates two wide-body jets, one 747.*



*Two columns assembled with bracing on weak axis to eliminate guying*

322 ft, with doors at the two 260-ft wide ends to allow entrance from either the north or south aprons. Roof trusses span 254 ft between the columns spaced 40 ft o.c. along the length of the hangar. The bottom chord of the truss is 78 ft above the hangar floor to clear the 65-ft high tail of a Boeing 747, the tallest wide-body aircraft, and to make room for overhead cranes. Hangar doors open 250 ft wide by 71 ft high. Mechanical utilities nest in the space created by the 25-ft depth of the trusses. Mechanical platforms and access ways are supported at the truss botton chord level to take advantage of what is normally unused space.

**Design of Truss**

Because of the short duration of start to completion of the project and the custom design of the roof-hung service platforms, establishing roof loads for design was critical. The architect's vast experience in the aircraft industry was useful in defining the service criteria required to quantify the

*Two adjoining trusses assembled on ground to avoid staging for assembly.*

loads for the structural design.

The roof truss is designed for the following loads: snow loads of 25 psf; truss self-weight, purlins, sway frames, top and bottom chord bracings—25 psf; heating and ventilating equipment, service platforms, draft curtains and other items within the truss depth—10.5 psf; crane system, two moving loads of 15,000 and 90,000 lbs. at adjacent panel points; and 7,600 lbs. at each runway rail support location.

A Pratt-type truss was chosen since it required less joints and consequently fewer field bolts. But the basic Pratt is not economical for a 25-ft deep, 254-ft span truss. The long, unsupported compression members became heavy. An innovative "triad frame" was introduced in the top panels of the conventional Pratt layout to provide lateral support in the weak axes of the top chords and vertical struts. This new arrangement helped reduce truss weight (each weighs 67 tons) by 15%.

**Lateral Stability**

The columns are fixed at the base in the major axis to resist lateral loads due to wind for this free-standing hangar. Wind forces on the 71-ft high hangar doors and the gable are transferred to the vertical bracings on the column lines by the truss bottom chord bracings. There were two choices for the foundation design—spread footing or drilled caissons. Estimates based on preliminary design indicated the caisson was more economical and faster than the footing option which required several subcontracts. A 6-ft diameter caisson, designed to resist base moment, was installed at each column.

**Erection Critical**

For this singly-bay, 105-ft tall building, stability of the structural elements during erection was very critical. Temporary staging and braces cost time and money. Structural details had to address erection methods for this fast-track, guaranteed-maximum project to be successful.

Columns, 100 ft tall at the cap plate level receiving the bearing detail of the trusses, did not require bracing in the strong direction. In the weak axis, however, they were extremely slender until lateral struts were installed. To avoid this unbraced condition of the erected column, two columns with vertical bracings and struts were installed completely on the ground so the assembly would be stable in both directions. The 40-ft wide assembly was then erected to eliminate the need for temporary guying. The set of eight anchor bolts at each column base had to be protected by special sleeves with tapered ends to permit lowering of the column boot in place without damaging the threads. The remaining columns were installed immediately by attaching the struts to the braced bays.

Truss elements were match-drilled for field-fit and shipped loose to the site because of the large truss size. To eliminate costly staging and erection, a set of two trusses was installed with sway frames, top and bottom chord bracings, platform frames and roof purlins at the ground level. The whole assembly, weighing in at 180

*Two-truss assembly placed on column cap plates. Erection took less than 30 minutes.*

tons, was lifted by two cranes and seated on the column caps for bolting. Erection of this massive grillage of structural elements, which went like clockwork, was completed in less than 30 minutes.

**Planning and Sequencing**

"Advanced planning, timely procurement and delivery of fabricated materials to the jobsite are the key words for the success of this fast-track project," said Chris Piotrowski, project manager for the Austin Company. "Once the basic design was completed, the engineering, purchasing, scheduling and construction groups of Austin maintained a close dialogue with the subcontractors until the job was done. Inputs from the fabricator and erector at the initial stage of the project contributed greatly to the smooth and timely completion."

Planning and teamwork produced the result. The first DC-10 rolled into the hangar on time and within the guaranteed maximum budget. □

**Architect/Engineer/Construction Manager**
The Austin Company
Des Plaines, Illinois

**Steel Fabricators**
Liebovich Brothers, Inc.
Rockford, Illinois, and
Steel Structures Corporation
Bethlehem, Pennsylvania

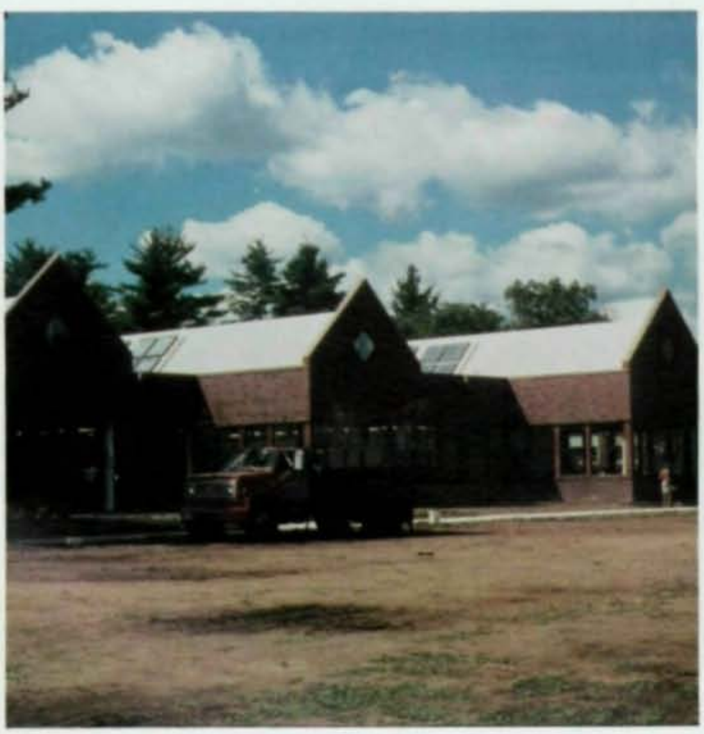
**Erector**
Broad, Vogt and Conant, Inc.
Gary, Indiana

**Owner**
United Airlines
Chicago, Illinois

*Ranjit Roy, S.E., is chief structural engineer of the Austin Company, Des Plaines, Illinois.*

# WESTFIELD STATE COLLEGE

## Dining in Record Time



The new Westfield State College dining hall is a good example of an imaginative collaboration between architect and structural engineer. A requirement of the new building was to permit a good deal of light into the dining hall and yet maintain a one-story building. Also, because this one-story building was to be surrounded by several multi-story buildings, the multi-gable roof system was adopted to avoid the building being dwarfed by its surroundings.

Originally, a clear-span framing system had been considered. However, because of the size and shape of the building layout, it became apparent a clear-span system was not the best solution. The alternate approach chosen was an exposed, multi-pipe truss framing system. The pipe-truss framing system covers approximately 14,000 sq. ft of public cafeteria area. Conventional steel framing covers the remaining 8,000 sq. ft of the building, an area to be used for kitchen and office use.

The framing system consists mainly of trusses. The 4-ft deep "joist trusses" frame

*(continued on p. 22)*

# HOW WE HELPE
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It's a single store with 222,250 square feet of space. And with everything from a supermarket on one side to a hair salon on the other, it's no wonder they call it "a mall without walls." Hypermart USA is a joint venture of Wal-Mart Stores, Inc. and The Cullum Company. Located outside Dallas,


*With 222,250 square feet, Hypermart USA marks the beginning of a new era in retailing.*

# D HYPERMART
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it's already a huge retailing success. But the real success story began well before they opened the doors. Because just building the structure was a major undertaking in itself—an undertaking Vulcraft was proud to be part of.

We supplied all 360 tons of steel joists and joist girders plus 250 tons of steel deck for the job. But more than that, we helped design the structure. That's because Wal-Mart brought together a special team to plan and execute the project—a team which included the architect, structural engineers, steel fabricator and Vulcraft as the structural system supplier. Each was a specialist Wal-Mart knew they could count on from experience. And their experience with Vulcraft was vast, since we'd already supplied materials for over 550 Wal-Mart stores.

On this project, we worked especially closely with the structural engineer to design the most economical bay spacing possible. Because on a job of this magnitude, every cost efficiency meant tremendous savings. Furthermore, throughout construction, we came through with 100% on-schedule sequential deliveries, making sure there were no expensive delays. This kind of planning and dependability are exactly why Wal-Mart has come to Vulcraft with so many jobs. And why you should be in touch with us, too, no matter what kind of spaces you need help with.

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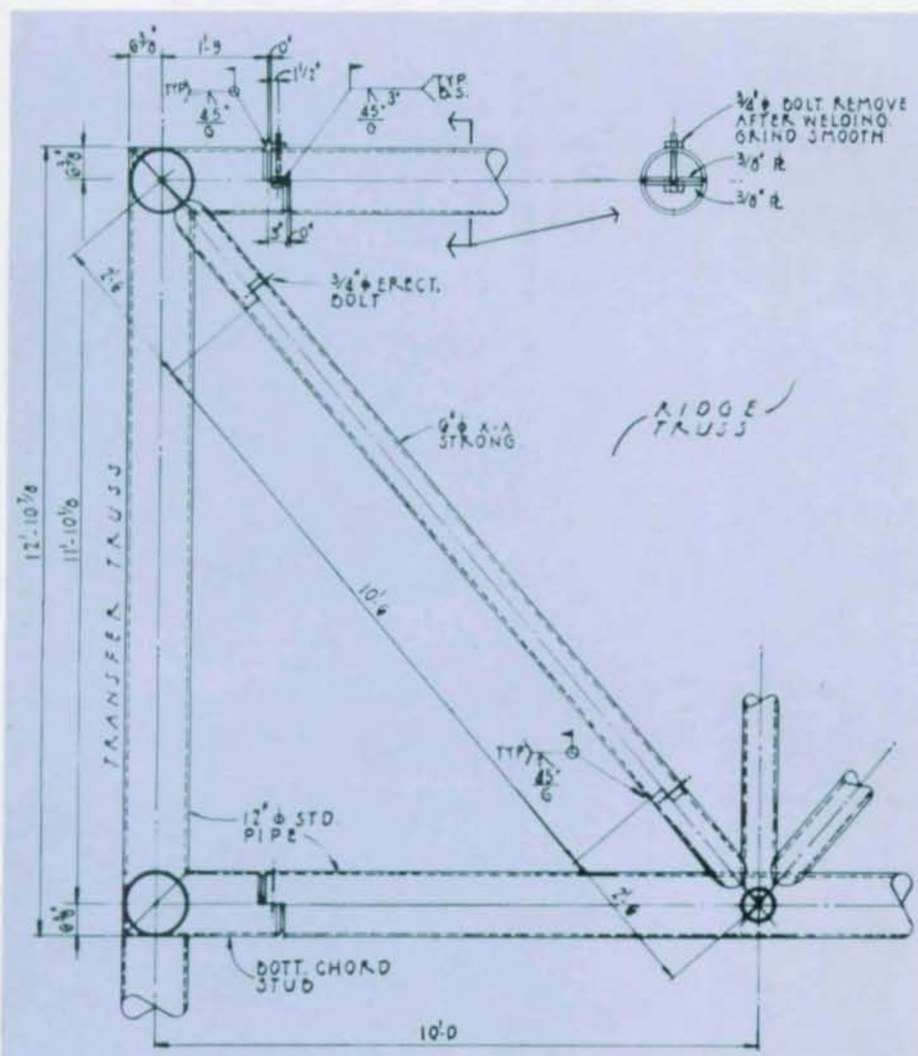
*Roof pipe-truss framing highlights dramatic cafeteria/dining hall. Column truss assembly detail below.*




to the 12-ft deep "ridge truss." Ridge truss spans range from 30 to 70 ft. The ridge truss, in turn, is attached to the 17-ft "transfer trusses" located in plan perpendicular to the ridge trusses. At the exterior edge of the building, the joist trusses frame to 4-ft deep "spandrel trusses." Finally, roof purlins are used to form the gable for the roof and truss braces complete the system. Three-in. accoustical roof deck is used in the entire dining hall area.

Generally, the pipe sizes used in the truss framing were dictated by architectural concerns. The ridge truss are 12-in. dia. top and bottom chords and 6-in. dia. verticals and diagonals. Truss members of the transfer truss are all 12-in. dia. pipes. All other trusses (joist and spandrel trusses) are 4 ft deep and consist of 5-in. dia. top and bottom chord and 3 in. dia. verticals and diagonals. In keeping with the established architecture, all other pipe columns were 12-in. dia. pipe.

The exterior window wall system is located off of the main pipe truss framing line. The wall system, 6-in. tube columns with 12-in. channels running horizontally, act as wind girts at the building eave line. To permit additional lighting in the open dining area, a pattern of skylights was added to the east side of the roof gables.

A great deal of thought went into the detailing of the pipe trusses to allow for ease in fabrication and erection. Typically, pipe truss members were shop welded using a flex-core semiautomatic procedure.

*Column truss assembly permitted speed of fabrication/erection which made "dining in record time" possible.*

However, the end diagonals of the ridge trusses were an exception. These diagonals were spliced for erection purposes. The ridge truss was spliced 2 ft from the end of the truss. The transfer truss, which is actually more like a vertical load column transfer system, came to the field in one piece. Included on the transfer truss were the 2-ft extensions necessary to receive the ridge truss. After the ridge trusses were placed, the final connection was made by field welding.

Total fabrication time for some 250 tons of steel pipe trusses was about six weeks. Setting of the structure was done in three weeks and final field welding was completed in a little over a month. The total job is scheduled for completion by September 1988. □

**Architect**
Juster Pope Frazier
Shelburne Falls, Massachusetts

**Structural Engineer**
Besier Gibble Norden
Old Saybrook, Connecticut

**General Contractor**
D. A. Sullivan & Sons Inc.
Northampton, Massachusetts

**Steel Fabricator/Erector**
Avenue Welding, Inc.
Middletown, Connecticut

**Owner**
The Commonwealth of Massachusetts

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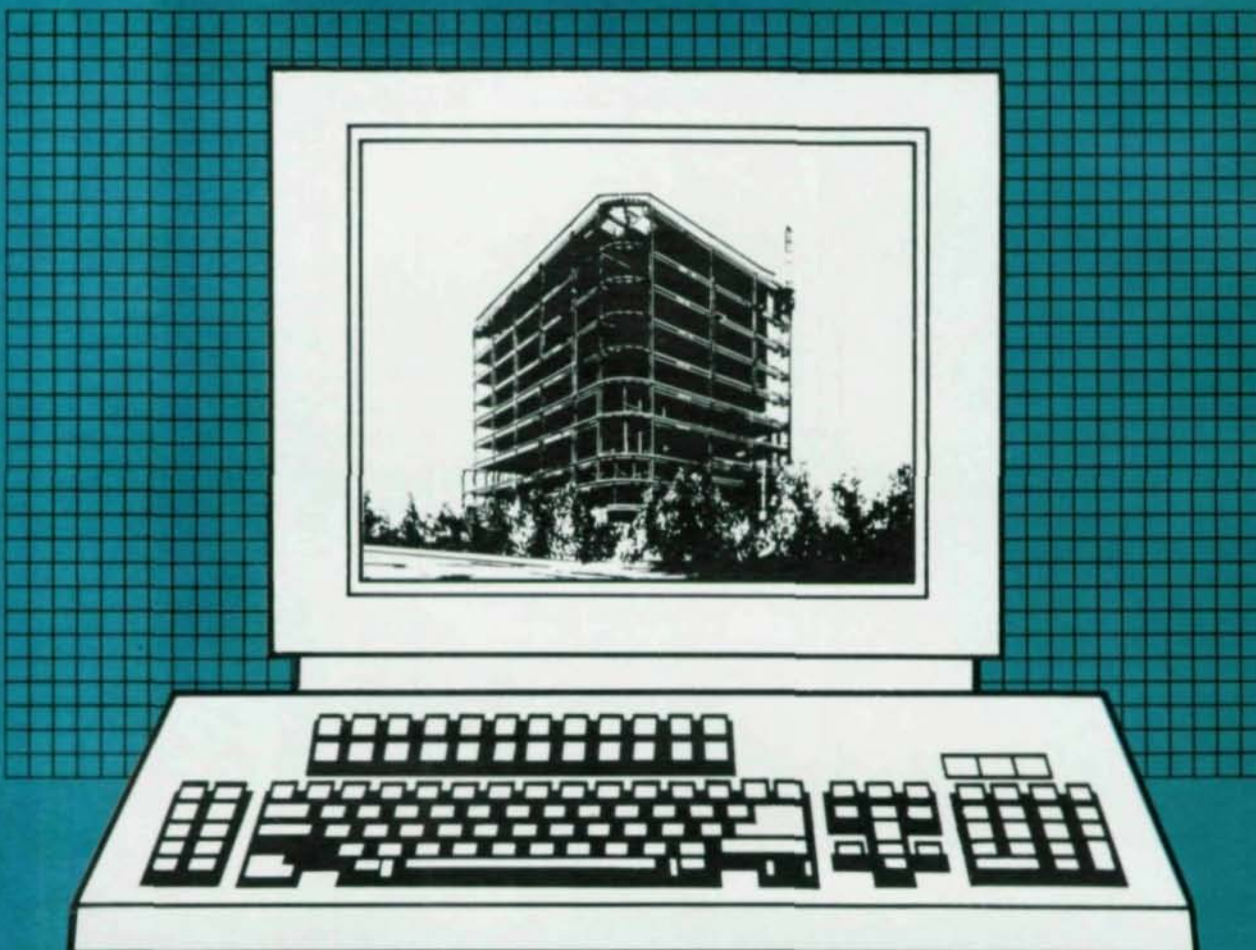

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# Special Bridge Section

## PEDESTRIAN BRIDGE

# St. Joseph's University Bridge

To passersby, the 85-ft long pedestrian bridge, with its arch and stone facing, appears as if it were transported from a quaint country setting to the middle of a major city thoroughfare. In reality, the stonework is only a facade supported structurally by fabricated steel plate girders. The walkway is part of St. Joseph's University's $40-million campus improvement and endowment of faculty chairs and scholarships. Other improvements at St. Joseph's, a 137-year old Jesuit university in Philadelphia, included the design and construction of a $10-million, 94,000-sq. ft residence hall. Done in a neo-Gothic style, the new dormitory will house nearly 300 students.

Part of the campus rests in Philadelphia, with the other section across the heavily congested City Avenue Highway in Lower Merion—hence the need for a bridge. During the school year, university officials anticipate 1,500 people daily will use the bridge between campuses.

In addition to obvious safety advantages, the university officials see the bridge as a symbol of change, providing St. Joseph's with more of a community presence. "Now, with the bridge in place, people can drive *through* our campus, instead of driving *past* it," Rev. Nicholas Rashford, university president, says.

**Bridge to the Present**

The cut limestone facade of the bridge is in keeping with the university's neo-Gothic architecture. Steel girders beneath the limestone facade provide support for the simple-span structure. Steel was chosen over precast concrete as more economical. Two main spans, 88 ft long and about four ft deep, are fabricated steel plate girders. And three smaller spans, 26 ft long, are rolled shapes. Between girder spans, rolled sections, or purlins, serve as floor beams.

The project manager, Dan Socket, explains that while the facade did not perform a structural function, it did influence structural design. "Because of the panels, our design criteria demanded limited deflection. Too much deflection would cause the panels to crack. We had to create a structural member with the section properties with the required strength and stiffness." He further indicated these constraints dictated the depth of the beam and that regular rolled steel shapes are not strong or stiff enough. Therefore, fabricated plate girders were designed to accommodate its span and loading conditions. The overpass runs from the new dormitory's second floor, to a stair tower and elevator pavilion—which is also a steel structure built on block-bearing walls—on the Philadelphia side.

**A Busy Night.**

City Avenue is a busy state highway, so to erect two 20-ton main sections would have been impossible during daytime hours. There was no alternative but to work by night. PennDOT granted the university a permit to close the state highway for the work from midnight to 8 a.m. Sunday morning. As luck had it, they issued the permit during St. Joseph's busy commencement weekend, when traffic would be even more congested.

"We had to work fast to keep traffic delays to a minimum, so all the structural steel had to be erected that morning," Socket said. "Commencement was an added time constraint since ceremonies were to begin only two hours after City Avenue reopened."

Workers were not out there alone. Students, parents, faculty and alumni—sensing the symbolic significance of the bridge—were also on hand to witness construction. One professor said, "I think this goes along with the idea to build up the image of the campus. All this new construction goes a long way toward letting people know we are changing—and should be seen as part of a commitment to provide the best education we can. Part of that is bricks and mortar—as well as books in the library." □

**Architect**
Sabatino Architects
Philadelphia, Pennsylvania

**Structural Engineer**
O'Donnell & Naccarato
Philadelphia, Pennsylvania

**General Contractor**
L. F. Driscol
Bala Cynwyd, Pennsylvania

**Steel Fabricator/Erector**
Samuel Grossi & Sons, Inc.
Bensalem, Pennsylvania

**Owner**
St. Joseph's University

## MOVABLE SPAN

# Route 88 Bridge Over Point Pleasant Canal

The new lift bridge on New Jersey Rt. 88 over the Point Pleasant Canal replaced a nearby low-level bascule bridge which had its piers in the canal. The tower piers of the new lift bridge are behind the existing bulkheads of the canal to provide unobstructed navigation for vessels and unrestricted flow in the heavily tidal waterway.

Available structural depth for the lift span was limited by a required 30-ft vertical clearance (in closed position) and the need to meet grades of local streets on both sides of the canal. A relatively close spacing of steel girders and a composite design with a lightweight concrete deck were selected as a solution to the problem. Plus, the superior safety and riding quality of a solid deck in comparison to open steel grating was deemed very important.

So as not to interfere with navigation, the lift span was erected in the open position with girders on temporary supports. Contract plans permitted two field splices per girder, but the contractor elected to fabricate girders in one piece. Transporting the 180-ft girders from plant to site required careful planning. But once on site, the one-piece girders enabled speedy erection since each unit was picked up by two cranes and placed on the temporary supports.

After the deck was poured—while the span was still on temporary supports—lifting ropes were installed, counterweights poured and the weight transferred to the sheaves.

Designers strived for a clean, uncluttered look for the structure, a prominent local feature. The 97-ft towers, which have few bracing members, are made up of 3-ft by 4-ft welded steel boxes. At the field splices of the tower legs, the splice plates are hidden inside the box section with only bolt heads visible.

The two sets of lift machinery, one on top of each tower frame, are totally enclosed in machinery houses, which appear integral with the towers. Counterweights, enclosed in painted steel boxes, also blend in. Two 125-hp motors raise the lift span to full open position in one minute. Cables and wiring for the machinery are routed to machinery rooms inside the tower legs to further enhance an uncluttered look.

**Designer/Structural Engineer**
Howard Needles Tammen & Bergendoff

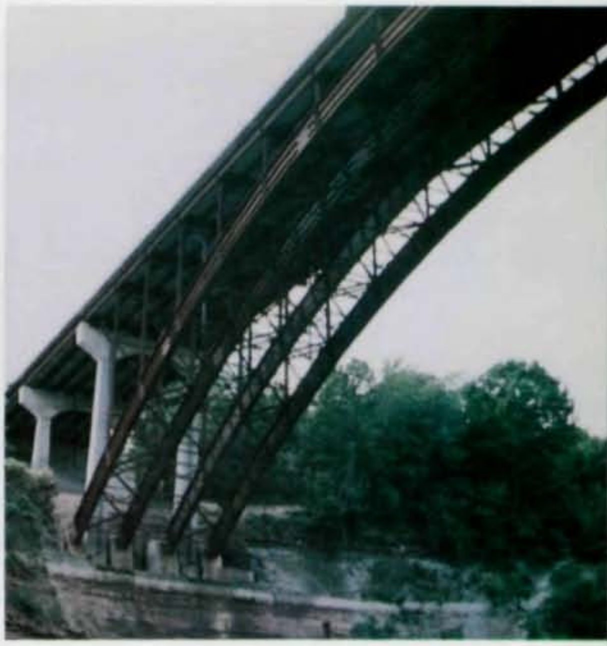
**General Contractor**
Raymond International Builders

**Steel Fabricator**
Harris Structural Steel Company, Inc.

**Owner**
New Jersey DOT

LONG-SPAN

# Driving Park Avenue Bridge



The Driving Park Avenue Bridge over the Genesee River is a deck arch 200 ft above the river bed. Built as a main span of the four-span (80+81, 436, 96) bridge, the steel arch is a two-hinged solid-ribbed type, with a span of 436 ft and rise of 74 ft (rise to span ratio 1:5.9). The bridge carries three 12-ft travel lanes, two 4-ft wide bikeways and two 8-ft sidewalks, with the reinforced concrete deck supported by a stringer and floor beam system. Rolled steel columns transmit the load from the floor beams to the four plate-girder arches on 18-ft centers. The parabolic steel arches are welded-plate girders with 96-in. deep webs. The bridge is designed for AASHTO's HS20-44 loading using Working Stress Method. All structural steel conforms to ASTM A588 weathering steel (unpainted).

Although various structural alternates were considered during the preliminary studies, it was concluded the steel arch and the slant-legged, rigid-frame structures were the most feasible choices. Conventional prestressed concrete and composite steel type construction, requiring a 200-ft high pier in the river were ruled out.

Steel box girders for the two alternate schemes also were considered. Steel box sections would involve erection of heavy members, further comduring fabrication. Steel box sections would involve erection of heavy members, further complicating the erection procedures presented by site conditions at the gorge. However, the placement of four plate-girder arches at 18-ft centers made the fabrication, erection and future maintenance much easier compared to two box-section arches. The whole system is more stable and redundant as compared to the non-redundant, two-arch system.

Where the approach spans demonstrate the smooth mixture of concrete substructure to steel superstructure, the main span expresses the best qualities of openness. Using a redundant number of slender steel arches, the potential qualities of structural steel are very well demonstrated. Using few supporting elements and responding to the demand for openness, the arch structure serves as a "frame" for the waterfall when viewed from the park. The pier profiles, adequate for a pleasing look, convey an arch effect. The span-to-rise ratio of the parabolic arch emphasizes the designer's effort to mold economy and esthetics.

Steel was erected by cantilevering from each skewback while anchoring to the abutments. Three hundred fifty-ton ringer cranes with a 280-ft reach were very adequately utilized. Provision was made at the skewbacks for adjustments during erection, thus eliminating any problem for the closing center segment.

Designer/Structural Engineer/Owner
New York State DOT
General Contractor
F. L. Ciminelli Construction Co.

**MOVABLE-SPAN**

# Rehoboth Avenue Bridge

The $8-million Rehoboth Avenue Bridge, open to traffic on August 1, 1986, was constructed as a replacement to an existing single-leaf bascule bridge. The new structure is a single-leaf, simple trunnion-type bascule bridge which carries Rehoboth Avenue over the Lewes and Rehoboth Canal in Rehoboth Beach, Sussex Co., Del. The importance of the crossing to this heavily beach-oriented community required provision for a temporary bridge during demolition and reconstruction since the new bridge is on the same alignment as the existing one.

The bascule span provides 50-ft horizontal clearance with 16-ft vertical clearance (span closed) and 35-ft vertical clearance (span open). The deck, 66-ft wide, carries two 12-ft traffic lanes, two 13-ft shoulders and two 6 ft-8 in. sidewalks, each with a 4 ft-6 in. high pedestrian/traffic rail. The roadway is open steel grating; the sidewalk is stiffened steel plate overlaid with epoxy/grit.

The bascule span has two welded steel bascule girders, spaced 47 ft-10 in., which support a stringer and floor-beam superstructure. The counterweight is a steel box filled with concrete framed between the girders. Each girder is supported on a simple trunnion bearing on a pair of welded-steel box trunnion towers. The forward part of the bascule leaf is stiffened by floor beams spanning the two bascule girders and a lateral bracing system which ties the girders together at the top and bottom flanges. The presence of the trunnion towers at the tail end interferes with the use of a bracing system between the bascule girders. To provide lateral stiffness, the bascule girders are made up of box sections at the tail end. Accordingly, each bascule girder is comprised of a single web and flanges in the toe section, with double web and flanges fore and aft of the trunnion. This innovative, transitional single-web girder/box girder detail was granted a 1988 Award for the Advancement of Arc Welding Design, Engineering and Fabrication by the James F. Lincoln Arc Welding Foundation.

**Designer/Structural Engineer**
Hardesty & Hanover Engineers

**General Contractor**
A & F. C. Wagman, Inc.

**Steel Fabricator**
Harris Structural Steel Company, Inc.

**Steel Erector**
Cornell & Company, Inc.

**Owner**
Delaware DOT, Div. of Highways

## MEDIUM-SPAN, HIGH-CLEARANCE

# Cochrane Bridge

The new Cochrane Bridge in Morgan Hill, Santa Clara County is in a mountainous area across Coyote Creek and reservoir. The roadway serves as the only public access to an isolated area of a few ranches, a county park and a state park. For 37 years, the previous bridge had undergone progressive distortion from being on the margin of the Calaveras Fault as well as from reservoir water fluctuations which caused landslides.

The new structure is a single-span, 300-ft long steel truss bridge, set 90 ft above the stream bed on cellular abutments. The steel truss structure is the type and weight to best withstand seismic-loading conditions. Also, transportation and erection of the smaller truss members could be achieved at this mountainous location with less difficulty than with a traditional plate-girder structure. The steel truss bridge was chosen over the traditional welded-plate, girder-type bridge because of its small depth from the roadway to the bottom truss chord as compared to the large depth of the composite, welded-plate girder design. This depth advantage permits use of existing roadway without using approach roadway fills which would have aggravated slope stability conditions.

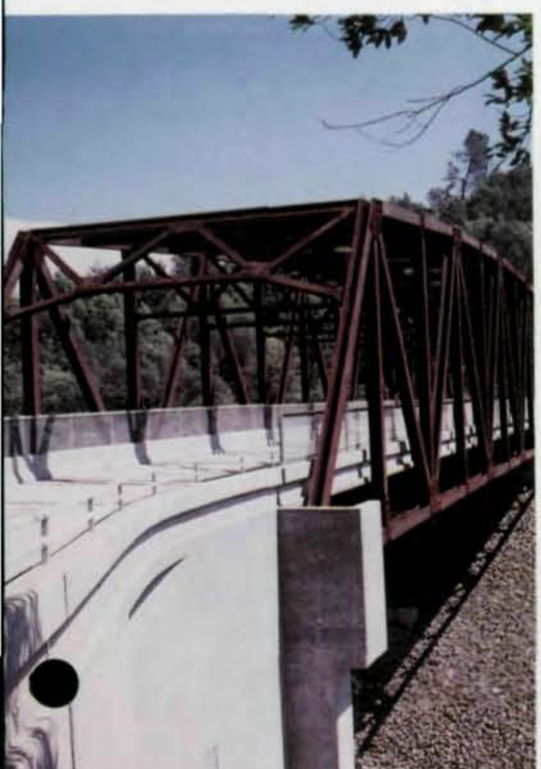
Unique features are the bridge foundations designed to resist earthquake forces of 7.5 (Richter scale) and reservoir earth slope and creep movements. The north abutment was constructed on coherent rock and has an earthquake restrainer attached to the rock capable of resisting 1.5 million lbs. of earthquake force. It will also allow rotation of the truss about the earthquake restrainer to provide for the anticipated movements of the south abutment. The steel-truss bearing supports and the south abutment, its design based on information gathered from monitoring the previous bridge, allow five feet of movement in the west and north directions, with an anticipated vertical movement of one foot. The south abutment slope has been designed with horizontal drains to remove ground water from the reservoir slope. Rock slope protection with a buttress rock toe was constructed at the south abutment to improve stability of its slope.

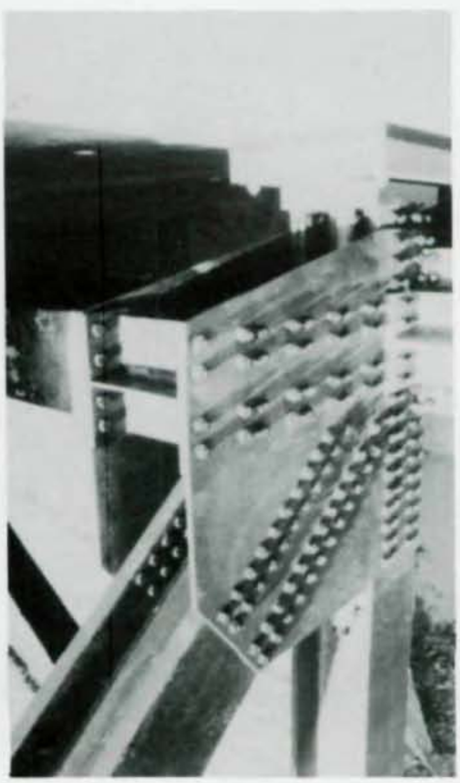
The truss design uses high-strength A588 structural (weathering) steel, resulting in a significant weight reduction, corrosion protection and savings in future painting. All truss members are welded steel-plate sections. To eliminate the need for fill plates, a constant depth of 16 in. was maintained throughout. The depth of the floor beams vary to accommodate a two percent deck cross-slope. All field connections are A325 high-strength bolts.

Designer/Structural Engineer
Taber Consultants, and
Wendell F. Pond, Bridge Design

General Contractor
Raisch Structures

Owner
Santa Clara County





MEDIUM-SPAN, LOW-CLEARANCE

# San Francisco River Bridge

The San Francisco River Bridge, Alma, N.M., is a single-lane, simple-span steel-plate girder structure 212 ft long, with a cast-in-place deck, 14-ft curb-to-curb wide. Abutments are cast-in-place concrete on spread footings partially cast in rock. All structural steel is high-strength, low-alloy, corrosion-resistant steel (weathering steel).

The San Francisco River here drains an area of 1,546 square miles. Design flow for $Q_{100}$ is estimated at 114,000 cu. ft per second; flow velocity at 24 ft per second. The existing crossing of this erratic river was a low-water concrete ford about a half mile upstream. It was destroyed on several occasions and not accessible for many weeks of the year. The site chosen for the bridge was a vertical-walled canyon 170 ft wide, 25 ft deep. Scour in the sandy stream bed was estimated at 15 ft. The river carries large debris during flood conditions.

The owner was concerned with the location of any piers in the channel because of the problems of debris, scour and magnitude and velocity of flood flows. The same concerns were expressed on the use of falsework in the channel for a cast-in-place concrete structure. With these concerns in mind, the designer proposed a simple-span, steel plate-girder structure. The single-lane bridge mandated a two-girder system. Economic considerations resulted in a longitudinally stiffened web, a cross-frame spacing of 23 ft and the elimination of wind bracing. Bolted splices were provided at the 1/4 and 3/4 points of the span to facilitate transportation and erection. All girder flanges and webs were considered fracture-critical members for this non-redundant structure and were fabricated and inspected accordingly.

The San Francisco River Bridge is the longest simple-span, steel-girder bridge in New Mexico. The bridge provides access to the Gila National Forest in New Mexico and Arizona for fuel wood gathering, recreation, grazing and fire control. The bridge is a minimal-maintenance, all-weather crossing of a large, erratic river—and is expected to serve the public for many years to come.

**Designer/Structural Engineer**
FHWA, Western Bridge Division

**General Contractor/Steel Erector**
A. S. Horner, Inc.

**Steel Fabricator**
Egger Steel Company

**Owner**
USDA, Forest Service

## SPECIAL PURPOSE-PEDESTRIAN

# Young Quinlan Skyway

The record-level pedestrian walking—commissioned by The 614 Company—is another link in the expanding skyway system of downtown Minneapolis. Linking the Young Quinlan Building to the Medical Arts Building and to the Minneapolis Skyway system, the bridge is a celebration of its crossing. Harkening back to the archetypal pedestrian bridges of Venice, the 80-ft long skyway seizes the opportunity to create and express a dynamic visual space.

The skyway not only links two buildings but also actively engages them. The Bridge rests on buttresses of honed and rusticated limestone which form portals into the street-level floors. At these portals the steel arch expression of the bridge merges with, and gracefully emerges from, the stone building's bases.

Skyways in Minneapolis become highly visible landmarks because the system requires them to play very active roles in the public domain. This bridge acknowledges its position in the city by creating a portal announcing the Nicollet Mall retail street and by enhancing the sympathetic resonance between two important historical buildings.

The simply supported steel beams literally link the richness of the Italian Renaissance palazzo of the Young Quinlan Building with the perpendicular Gothic of the Medical Arts facility.

Designer/Structural Engineer
Ellerbe

General Contractor
PCL, Minneapolis

Owner
The 614 Company

## MSC ADVERTISERS

## SHORT-SPAN, RECONSTRUCTED

# Clark Avenue Ramp

The Clark Avenue Ramp, a recently completed three-lane facility, provides access to the industrial flats from the west bank of the Cuyahoga River Valley. Its removal was a loss of a major valley crossing and direct access to an important industrial area. The ramp is a 976-ft long steel viaduct-type structure and a mechanically stabilized embankment (reinforced earth) about 130 ft long. The roadway, providing for one lane of traffic each way, plus a climbing lane for slow-moving vehicles, has a total width between deflector type barriers of 40 ft.

The viaduct structure has a continuous rolled-steel beam superstructure of nine 80-ft spans alternating with eight 32-ft spans. End-welded steel stud shear connectors ensure the reinforced concrete deck acts compositely with the six lines of steel stringers spaced 10-ft o.c. The superstructure is supported by a reinforced concrete abutment at each end and steel towers at the intermediate support locations. The steel towers reduce in height with the change in ramp grade.

Because early completion of this project was so important, a comparison study was made of steel and concrete towers to determine which allowed the earliest completion date. Although the schedule indicated it was possible to construct either alternate in the time available, it became obvious the concrete was more dependent on weather and other uncontrollable contingencies and that the advantage was to use the steel alternate. In addition, the concrete alternate would weigh at least three times as much as steel, thereby imposing more dead load on existing foundations.

The towers are four-legged structures connected by longitudinal and transverse struts and capped by two transverse girders to provide seats for the superstructure beams. All tower members are hollow boxes fabricated from steel plate, with all shop connections welded. The towers were designed to maximize shop fabrication, to minimize field connections and to facilitate shipping and erection. All field connections, consisting of joints at each end of the struts and the tops of tower legs were field-welded. Shop welds were tested selectively (25% minimum) with ultrasonic flaw detection. Field welds were 100% tested. The box sections were designed to be air-tight to eliminate internal corrosion, and reduce maintenance.

Viewed from a distance, the superstructure of the new Clark Avenue Ramp forms a smooth curve from the west bank of the valley across the tops of the towers to merge with the reinforced embankment at the east abutment. The curve of the deck depicts graphically the function of the structure. The towers, the dominant structural elements of the ramp, present an obvious appearance of strength and stability. The box sections provide smooth, uncluttered surfaces to form a structure pleasing in appearance and easy to maintain. And, in keeping with the urgency of the situation, the ramp was finished on a very strict schedule—and welcomed enthusiastically by both the local citizenry and the commercial interests in the flats.

Designer
Stilson & Associates, Inc.
General Contractor
The Horvitz Company
Steel Fabricator/Erector
Conn Fabricating & Engineering Co.
Owner
City of Cleveland

# Southwest Interceptor Crossing

The crossing of the Cuyahoga River Valley with the Southwest Interceptor sewer has been discussed and analyzed since 1969. Past studies investigated various aerial structure and siphon alternates. The crossing required traversing both the Cuyahoga River and the Old Ohio Canal along a line about 150 ft south of an existing ConRail bridge. An initial study on preliminary design of an aerial crossing was made in 1969 and it indicated the valley crossing would consist primarily of steel pipe supported on pier structures spaced at a maximum of 40 ft. Crossing the river and canal, with required spans from 100 to 330 ft, alternate methods of crossing were considered. These alternates included steel truss, arch and cable-stayed girder structures. Based on cost considerations, the steel truss bridge was selected as the design recommendation.

In a 1982 Facilities Plan, as in other studies comparing the aerial structure to a siphon, the aerial structure was the recommended basis of design based on costs related to construction, operation and maintenance, anticipated construction problems, impact on the Cuyahoga River Valley and system reliability. On the basis of all considerations, an aerial structure became the most logical solution to cross the Cuyahoga River Valley. The profile of the aerial structure carrying the gravity sewer is controlled by the required vertical clearance for crossing an existing B & O Railroad track at the west side of the valley and the elevation of the junction chamber inlet at the Southerly Wastewater Treatment Plant to the east of the crossing. Along the profile, the width of the Cuyahoga River and Old Ohio Canal determine the span lengths at each of these crossings. The recommended profile for the aerial structure is based on maintaining adequate design velocities in the gravity sewer, determined to be about 0.0032 ft/ft.

The total length of the valley crossing is in excess of 1,100 ft, with geometric constraints for horizontal and vertical clearance at the railroad tracks, river and the canal. Various types of structures were considered to support the sewer where it spans the Cuyahoga River.

Steel construction has inherent advantages considering weight and ease of construction for superstructures of this span length. Economic studies for the bridge types considered resulted in the following cost ratios, assuming the truss bridge as unity:

| | |
|---|---|
| Steel truss | 1.00 |
| Steel-tied arch | 1.25 |
| Steel-plate girder | 1.08 |

Judged on the basis of the estimated costs, the aerial structure was most advantageous using 60-ft self-supporting steel pipe spans, with a truss bridge spanning the Cuyahoga River.

Aesthetic considerations also have been addressed since the first report in 1969, making it apparent the project location is very sensitive. The first existing bridge is a combination truss and steel girder structure that carries the previous Big Creek Interceptor. The second is a trestle bridge for the ConRail tracks. The truss structure ultimately selected for the interceptor blends exceedingly well with the existing environment. It has the added advantage of clearly delineating the advances in truss bridge design and aesthetics occurring since these earlier structures were built. The tubular steel truss members, chosen for their smooth, clean appearance are most advantageous for design purposes. The structure was analyzed as a space frame. And the bending and torsional stresses caused by three-dimensional loadings, including the very significant vertical and longitudinal loads of the 114-in. pipe, are assumed efficiently by the tubular framing members. The simplicity of the framing connections enhances the overall appearance and adds to the aesthetic impact of the bridge.

**Designer**
Stilson & Associates, Inc.

**General Contractor**
The Albert M. Higley Co.

**Steel Fabricator**
Commerce Steel, Inc.

**Owner**
Northeast Ohio Sewer District

SHORT-SPAN

# Gunpowder Falls Bridge

The existing Maryland Route 7 bridge over the Gunpowder River in Gunpowder State Park, Md. was constructed in 1924. The 120-ft reinforced concrete arch with a 24-ft roadway was functionally deficient. It had deteriorated severely even though it had been rehabilitated in 1972. Since most of its main structural members were cracked or spalling, the decision was to replace the structure.

The natural valley setting of the park, frequented by hikers and waterway tubers, dictated a special structure. The decision was made to duplicate the existing bridge's waterway opening so as to minimize the amount of hydrologic/hydraulic design required by the Maryland Water Resources Administration. This shortened considerably the amount of design time required for a replacement structure.

A number of alternates were considered, among them several steel girder options with conventional concrete piers, until the painted steel arch was chosen. Although a painted steel arch would cost more than a conventional steel-girder structure, it was decided aesthetics governed, and the slender lines of the arch in the valley presented the Highway Administration with a classic situation for this type of structure. The replacement structure has a 124-ft arch span (227 ft c. to c. bearings) with a 32-ft roadway (two 12-ft lanes and two 4-ft shoulders).

The existing bridge was removed by explosives, which dropped the arch in the waterway in three large sections after the deck and columns were removed. This minimized the amount of in-stream use, since Gunpowder Falls is a trout stream, and it did minimal damage. Each ring of the proposed steel arch was constructed on land in two segments and supported by cranes for splicing, again with limited in-stream activity.

To eliminate impacts to Gunpowder State Park, it was decided to build the new structure in the same location as the existing arch, within state-owned right-of-way. Because a fast completion time had been promised local residents—eight months—they were content with the temporary inconvenience of some detours. The speed of fabrication and erection of the steel arch more than offset the higher cost.

Close coordination between design, construction and environmental firms and agencies satisfied the needs of the highway facility, the public and an environmental area in all its phases.

**Designer/Owner**
Maryland State Highway Administration–Bureau of Bridge Design

**Structural Engineer**
Envirodyne Engineers

**General Contractor**
Central Atlantic Contractors

## STEEL NOTES

### FOR YOUR CALENDAR

The 1989 National Steel Construction Conference is scheduled for June 21-24, 1989 at the Opryland Hotel, Nashville, Tenn. The Opryland Hotel is a popular site which also accommodates heavy exhibit equipment. Organizers expect 100-125 exhibitors this year and over 1,000 industry professionals to attend. Special workshop sessions are planned to focus on the specific interests of fabricators, consulting engineers and educators.

The 2nd National Symposium on Steel Bridge Construction will be held at The Shoreham Hotel, Washington, D.C. Oct. 19 & 20, 1989. The day-and-a-half program will offer qualified speakers presenting current practices and future advances in steel bridge construction and the co-relation of design techniques with fabrication, detailing and erection principles.

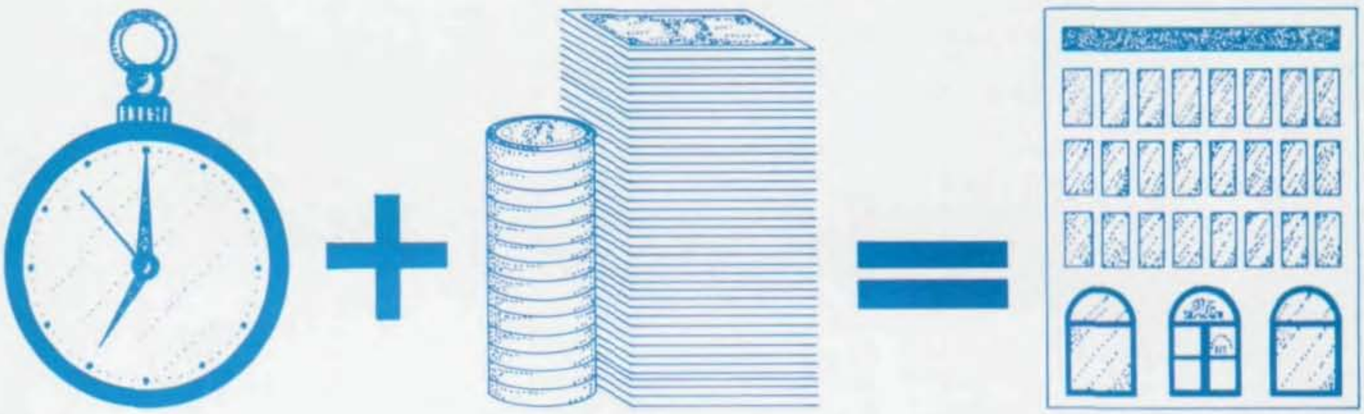
### OUT-OF-PRINT BOOKS AVAILABLE

Limited amounts of AISC publications no longer listed in the AISC Publications List are now available while the supply lasts.

*Simple-span Steel Bridges—Composite-beam Design charts* (T110) is a design aid that permits the direct selection of the most economical combination of rolled beam and coverplate acting compositely with a concrete slab as a component of a simple-span bridge to support HS20 highway loading. Separate sets of design charts are provided for 36 ksi and 50 ksi steels. The charts conform to the provisions of the 1973 AASHTO specifications. The book, produced in 1969, 176 pp., costs $12 (no discount).

To order any of the books listed above, send check, money order or Visa/MasterCard information (state type of card, number and expiration date) to AISC Publications Dept., P.O. Box 806276, Chicago, Ill. 60680-4124.

# KEEP TRACK OF YOUR BUCKS —WITH BUCKTRAK



AISC Marketing, Inc., was formed in 1986 by the structural steel industry. Its goal is to assist engineers, contractors, construction managers, architects and others in preparing proposals which represent optimum designs for steel framing for buildings and bridges. Now in its third year—sponsored by the American Institute of Steel Construction, Bethlehem Steel Corporation and the United States Steel Division of USX Corporation—AISC Marketing maintains a headquarters engineering staff and a national network of regional engineers to bring information about cost-efficient structural steel framing for buildings and bridges to their target audiences.

Simply by contacting the local AISC Marketing regional engineer, consulting engineers, DOT officials, contractors and others who make decisions on structural systems and materials for buildings or bridges can receive AISC Marketing's ideas and recommendations with a Preliminary Design Study. The Study may offer a single design which represents a "best" design. Or, it might include a number of ideas to show a range of values within the structural and functional parameters of the project. All recommendations are aimed at helping the engineer make the best design decision.

However, in building design, the structural and functional considerations may be superseded by more than a first-cost evaluation. From the developer's standpoint, income generated by the building is its only reason for existence. On the other side of the ledger is the problem of cost—both capital and operating. If lower cost is desired, as it usually is, steel framing contributes to that end in a variety of ways.

The cost of a structural steel frame is affected by three principal factors:

**Mill Cost.** Obviously the designer plans to use a minimum amount of steel. However, he also recognizes there may be a practical limit to weight reduction. The complexity of his design—the fabrication and/or erection complications must be considered. Thus, a search for a minimum weight solution may be self-defeating. There is also the problem of the correct application of high-strength, low-alloy steels, which for a slightly higher price, reduce the tonnage of steel required.

**Fabrication Cost.** Initial discussions between designer and fabricator must include consideration of experience, capacity and shop time. The fabricator sells man-hours. That cost includes wages, overhead and profit. A well-thought-out project saves a lot of shop cost.

**Erection Cost.** Again, the principal factor is time. But, here structural systems interface directly with other systems in the building. At this point, the designer considers the effect a steel frame and its erection sequence has on all other construction costs.

Interesting to note, even though structural steel requires careful planning and scheduling for the various steps of material ordering, rolling, shop drawing, fabrication and erection, it is probably the most commonly used structural material when time is the paramount consideration. The great advantage is the potential to make changes during design, fabrication or erection sequences with a minimum of increased cost.

Added to these direct construction costs is an indirect cost that often means the difference between "go" and "no go" on a project. Some years ago, MIT prepared a study on the cost of building projects. They based their study on the analysis of two areas of costs: direct construction costs, bricks and mortar and non-construction costs—land, fees, general administration *and* the cost of construction money. The study concluded non-construction costs amounted to 39% of the total project, and time-dependent (variable) costs 16%. The largest of the variable costs was construction loan interest, which amounted to 8.1% of the total project cost. (In this study, the loan cost was based on a 6% rate).

According to the study, three things affect loan interest costs: 1. the length of construction time, 2. the rate of the construction loan and 3. the pattern of construction spending. If the expenditure of construction monies, including construction interest, is plotted against time of construction, the result is a cash-flow curve that progresses from the origin, where both time and money are zero, upward to the point where the building is complete and the construction loan paid off with permanent financing. The analysis of that curve and the factors defining it can be interesting and helpful to the developer, since there are many strategies both he and his contractor can invoke to reduce those interest costs.

If the cash flow for a particular construction scheme can be estimated, the computation of the points on that curve is not too complicated. However, there is no simple program availabe that is organized for engineers or contractors. Lack of such a program was highlighted at the 1988 AISC National Steel Construction Conference. In his keynote address, Walter P. Moore of Walter P. Moore & Associates, Houston, Tex. stressed that "time is money." In his presentation, which compared various structural schemes for the same building, he made the point that each had a distinct and predictable time of construction. While the capital cost of each system presented could be estimated, he was unable to evaluate the differences in the times of construction. Several times he stated, "I wish we had a computer program that takes into account the time cost of money for a project."

AISC Marketing now has such a program to fill that wish—it is called BUCKTRAK. With BUCKTRAK, AISC Marketing is able to compute the construction interest cost for a building. But, keep in mind, BUCKTRAK is *not* an estimating program. It is simply a system for figuring interest cost from input provided by the contractor, construction manager or owner.

When using BUCKTRAK, three questions must be answered. First, what are the expenditures for the project? Second, when and at what rate will they be spent? Third, what is the assumed interest rate for the construction loan? Interestingly, for any given project, BUCKTRAK will evaluate several rates of interest if the construction loan rate can only be estimated. This type of program is not unique, but AISC Marketing has organized the input so it is understandable and convenient for contractors, construction managers or owners in comparing different construction schemes.

This program is similar to one the Canadian Steel Construction Institute instituted with this type of information for contractors and engineers in Canada. With their program as a starting point, AISC Marketing created this version.

In this issue of *Modern Steel Construction*, a tear-out insert can be completed by those who wish to use this service. Should you wish to use this service to determine the effect of the time and rate of construction expenditures for competing systems—follow these directions:

**First,** contact your local AISC Marketing regional engineer (addresses and telephone numbers attached) or call Ronald L. Flucker to determine if you have the right information for the estimate. What is needed is a complete construction schedule for the different segments of the building, along with costs of these elements.

**Next,** make copies of the form and fill in one form for each construction scheme. Starting with input Page A, complete the blanks at the top of the page following "project," "scheme," and "total elapsed months to completion" for this scheme. Next, enter the low interest rate assumed, the high rate assumed and the increments to be studied within the range. For instance, if you enter 12, 14 and 0.5 the computer will run the study for 12.0%, 12.5%, 13.0%, 13.5% and 14.0%.

This page of the form provides a good guideline for the elements and schedule of a steel-framed building. Forty-four activity names are shown in the column under "default activity name." These will show up on the printout, unless they are changed by entering new activity names in the column under "revised activity name." Therefore, there can be as many as 44 categories of activities input for any one project. If the designer compares the construction of a steel building with one of concrete, quite a few activity names will have to be revised for the concrete.

Input Page B of the form has the corresponding 44 activities shown vertically and 18 months horizontally. However, the program allows 36 months of input, so Page B must be reused for construction periods exceeding 18 months.

There are two ways to indicate the amount to be expended for an activity. To simplify data entry, if the expenditure is spent *equally* over a number of *contiguous* months, under "amt" enter the total amount expended for that activity. Then under "ml" enter the number of the first month of expenditures and under "#m" the number of months it will take to spend it. Entering "150, 2, 3" means that $150,000 will be spent on that activity in equal increments for 3 months, $50,000/month, starting in the 2nd month. If, however, the amounts to be spent on an activity are *non-contiguous* or *unequal*, varying from month to month, enter the amount to be spent under each month.

That is all it takes! It is just that simple. The completed form can be mailed or Faxed 412/394-3710 to AISC Marketing in Pittsburgh. Your data will be inputted—and within a day the answers will be on their way back to you.

It should be noted—this is a computational service only. AISC Marketing does not provide construction cost estimates, although the regional engineers will help contractors, construction managers and engineers develop realistic cost estimates for the activities associated with a steel-frame. These include such items as fabrication, erection, deck, fire protection, fill, etc. Also, a Preliminary Design Study can be requested to help develop the most cost-effective structural steel frame. The study also pinpoints other important construction information, such as foundation loads and where a steel frame may lead to other economies. □

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- AISC Marketing, Inc. - BUCKTRAK: Project Cash Flow Analysis input page A

*project: ______________________________ *scheme: ______________________________

*total elapsed months to completion:__ *nominal % interest rates per annum: low=__._ high=__._ inc=_._

*values in K$

| a# | <---------default activity name--------> | a# | <---------revised activity name--------> |
|---|---|---|---|
| --->indirect | | | |
| 01 | LAND | 01 | ______________________ |
| 02 | PROFESSIONAL FEES | 02 | ______________________ |
| 03 | TAXES | 03 | ______________________ |
| 04 | PERMITS | 04 | ______________________ |
| 05 | MISC | 05 | ______________________ |
| --->overhead | | | |
| 06 | STAFF ON SITE | 06 | ______________________ |
| 07 | SITE FACILITIES | 07 | ______________________ |
| 08 | HEAT & PROTECTION | 08 | ______________________ |
| 09 | HEAD OFFICE | 09 | ______________________ |
| 10 | MISC | 10 | ______________________ |
| --->substructure | | | |
| 11 | SITE PREPARATION | 11 | ______________________ |
| 12 | BULK EXCAVATION/CORE | 12 | ______________________ |
| 13 | BULK EXCAVATION/REMAINDER | 13 | ______________________ |
| 14 | CORE FOUNDATION | 14 | ______________________ |
| 15 | CORE/FOUNDATION TO GRADE | 15 | ______________________ |
| 16 | COLUMN FOOTINGS | 16 | ______________________ |
| 17 | WALL FOOTINGS | 17 | ______________________ |
| 18 | STRUCTURAL STEEL/FOUNDATION TO GRADE | 18 | ______________________ |
| 19 | SLAB ON GRADE | 19 | ______________________ |
| 20 | BASEMENT WALLS | 20 | ______________________ |
| 21 | GROUND FLOOR STRUCTURE | 21 | ______________________ |
| 22 | PARKING & CONCOURSE FLOORS | 22 | ______________________ |
| 23 | MISC | 23 | ______________________ |
| --->superstructure | | | |
| 24 | CORE/GRADE TO ROOF | 24 | ______________________ |
| 25 | STRUCTRUAL STEEL/GRADE TO ROOF | 25 | ______________________ |
| 26 | STEEL DECK & CONCRETE | 26 | ______________________ |
| 27 | SPRAYED FIRE PROTECTION | 27 | ______________________ |
| 28 | EXTERIOR CLADDING | 28 | ______________________ |
| 29 | INTERIOR PARTITIONS & DOORS | 29 | ______________________ |
| 30 | FINISHES | 30 | ______________________ |
| 31 | MILLWORK & EQUIPMENT | 31 | ______________________ |
| 32 | PLUMBING & DRAINS | 32 | ______________________ |
| 33 | HVAC | 33 | ______________________ |
| 34 | SPRINKLERS | 34 | ______________________ |
| 35 | ELECTRICAL POWER & LIGHTS | 35 | ______________________ |
| 36 | COMMUNICATIONS & ALARMS | 36 | ______________________ |
| 37 | ELEVATORS & STAIRS | 37 | ______________________ |
| 38 | MISC | 38 | ______________________ |
| --->general | | | |
| 39 | PAVING & LANDSCAPING | 39 | ______________________ |
| 40 | DESIGN CONTINGENCIES | 40 | ______________________ |
| 41 | POST CONSTRUCTION CONTINGENCIES | 41 | ______________________ |
| 42 | DEMOLITION & EXCAVATION | 42 | ______________________ |
| 43 | ROOFING | 43 | ______________________ |
| 44 | MISC | 44 | ______________________ |

input instructions:

========================================

*Fill out at least two sheets, one for concrete and one for steel. For concrete, activities (a#) 18,25,26, & 27 must be renamed. In general, all a# can be renamed and the default names are to serve as a user checklist. To have a blank a# name, enter "blank".

*page A:
- *max months = 36
- *if only one interest rate, enter as low value
- *inc = increment between high and low

*page B:
- *for an activity, enter cost by month and/or as a lump sum (amt)
- *for lump sum:
  - m1 = month #1 of cost
  - #m = # months of cost
- *if months over 18 are needed, use third page and circle month "19" on that sheet

*values in K$

```
                 |<----------------------------------------elapsed months---------------------------------------------------->|
                 |--19-|--20-|--21-|--22-|--23-|--24-|--25-|--26-|--27-|--28-|--29-|--30-|--31-|--32-|--33-|--34-|--35-|--36-|
a#|--amt-|m1|#m|--01-|--02-|--03-|--04-|--05-|--06-|--07-|--08-|--09-|--10-|--11-|--12-|--13-|--14-|--15-|--16-|--17-|--18-|

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                 |--01-|--02-|--03-|--04-|--05-|--06-|--07-|--08-|--09-|--10-|--11-|--12-|--13-|--14-|--15-|--16-|--17-|--18-|
                 |--19-|--20-|--21-|--22-|--23-|--24-|--25-|--26-|--27-|--28-|--29-|--30-|--31-|--32-|--33-|--34-|--35-|--36-|
```

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