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CONTENTS

EDITORIAL

On August 29th, impressive dedication ceremonies opened the lower level of the fabled George Washington Bridge (see story page 8). Highlight of the gala was the unveiling of a commemorative bronze bust of Othmar H. Ammann, designer of the bridge and consultant for Ammann & Whitney on the construction of the lower level.

In accepting the honor, Mr. Ammann said:

"Permit me to express in a few words my deep appreciation of the great honor you have conferred upon me.

"I accept it with humility, but with a certain embarrassment, because it is not usual that an honor of this kind is conferred on a man during his lifetime.

"In accepting this honor, I am also remindful that the technical accomplishment for which it is conferred, the building of the George Washington Bridge, is that of an engineering organization with which I must share the honor.

"My greatest satisfaction is derived from the fact that the George Washington Bridge has proven successful and of great value to the communities in the States of New York and New Jersey and to the traveling public. I am confident it will continue to do so under the splendid administration of that great public institution, The Port of New York Authority, which has created it.

"I thank you."

Mr. Ammann's brief speech, sharing the honors with others, is liberal evidence of the innate modesty of this great bridge engineer who has designed some of the country's largest and most beautiful bridges. As proof of his great influence in improving the aesthetics of bridges, many of those with which he has been connected have been awarded prizes for architectural excellence by the American Institute of Steel Construction.

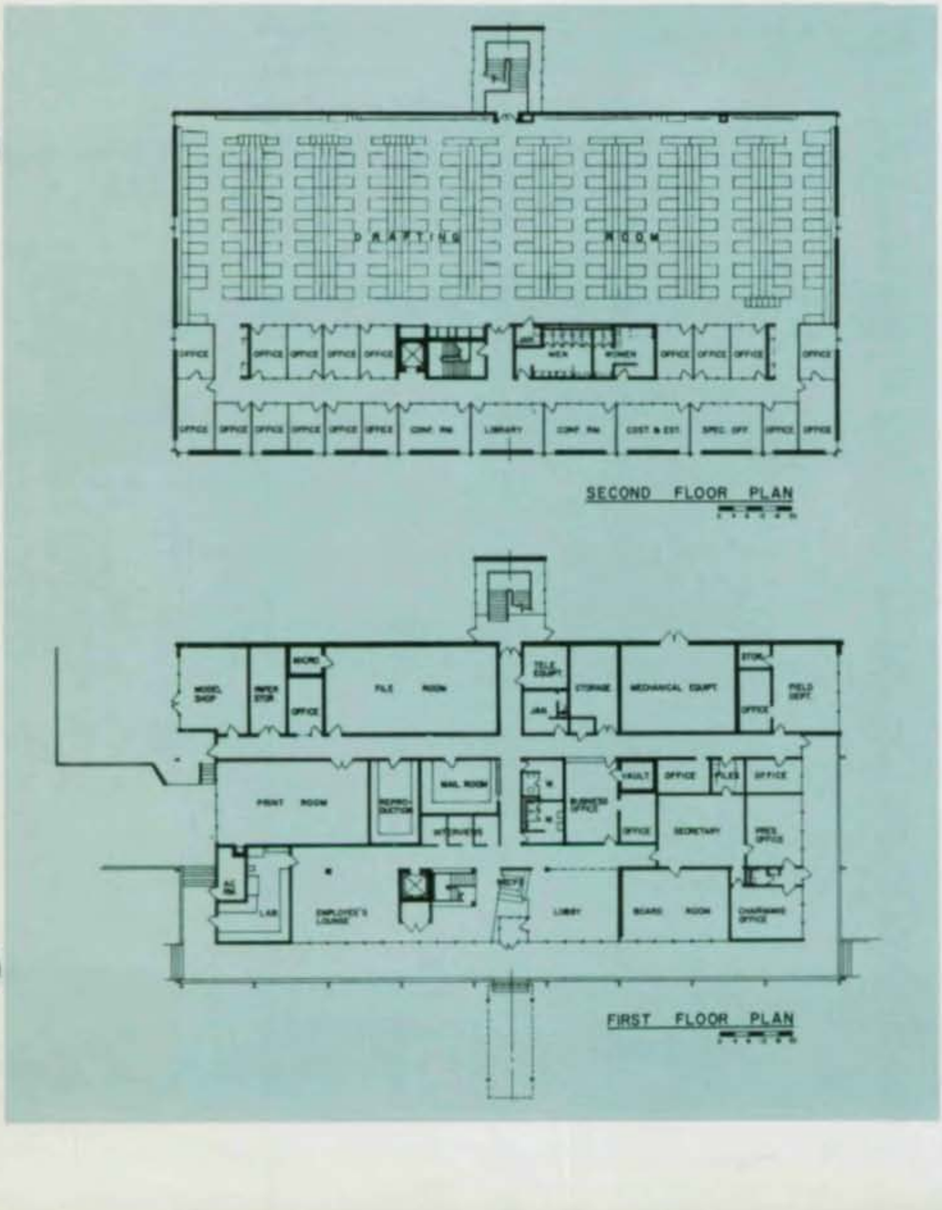
ENGINEERING FIRM DESIGNS OWN BUILDING IN STEEL

by E. E. Hanks, Technical Editor

When J. E. Sirrine Company, engineers-architects, of Greenville, South Carolina, designed their own office building, the choice of material for the frame was exposed structural steel. Designed especially for the needs of an architectural and engineering organization, this office building, completed and occupied early this year, includes many unique features.

The exposed structural system of steel expresses the feeling of strength, durability and clean orderliness, and combined with ease of erection and maintenance satisfies building code as well as design requirements. The structural system, expressed on the exterior, furnishes visible, yet no massive, support for the second floor, which cantilevers over the first floor and provides covered walkways with solar protection for the lobby, executive offices and employees' lounge located on the first floor. Service spaces, such as filing, reproduction and printing, are also on this floor.

The second floor contains the major design and production facilities, including a drafting room for 130 architects, engineers and draftsmen, with office spaces for department heads, project managers, secretaries, and library and conference rooms. Each of the 130 spaces for architects, engineers and draftsmen is a separate and distinct area, being separated by visual dividers constructed of steel tubes and pegboard with a built-in reference table and bookcases also supported on steel tubes. The space also contains adequate cabinetwork for storage of equipment and personal supplies. Only slender steel legs of the built-in equipment are visible below desk height, thus providing an uncluttered feeling and easy maintenance.



The light and airy feeling is further accented in the drafting room through the use of a louvered plastic ceiling, above which is located the lighting and air conditioning ducts. This open and unencumbered space is accomplished by the use of long span joists supporting the roof deck.

The partitions between offices and corridors on the second floor are constructed of slender steel partition members to further accent the feeling of openness.

The project, involving 32,500 square feet of building on a three-acre plot, includes parking for over 100 employee cars, visitors' parking spaces and complete landscaping including a formal garden on the north side of the building. The executive areas on the north side of the building are tastefully decorated with warm materials, such as carpeting and cherry paneling, which further complement the permanency and strength of the basic materials used.

Total construction and site development costs are reported to be in excess of one-half million dollars.

Total weight of the structural steel frame and long span joists is 115 tons, or seven pounds per square foot of floor area.

Architects and structural engineers, J. E. Sirrine Company; general contractor was Yeargin Construction Company. Structural steel was fabricated by Greenville Steel and Foundry Company, all of Greenville, South Carolina.

THE MOST BEAUTIFUL BRIDGES — 1961

LONG SPAN PRIZE BRIDGE WITH MAIN SPANS OVER 400 FEET
Sherman Minton Bridge, Highway I-64, crossing the Ohio River between New Albany, Indiana, and Louisville, Kentucky
Owner: Indiana State Highway Commission and Kentucky Department of Highways
Designer: Hazelet and Erdal
Fabricator: The R. C. Mahon Company

MEDIUM SPAN PRIZE BRIDGE WITH FIXED SPAN UNDER 400 FEET AND COSTING MORE THAN $500,000
New Sixty-Second Street Bridge, Pittsburgh, Pennsylvania
Owner: State Highway Department, Commonwealth of Pennsylvania
Designer: Parsons, Brinckerhoff, Quade & Douglas
Fabricator: American Bridge Division, United States Steel Corporation

MEDIUM SPAN BRIDGE, AWARD OF MERIT
Randolph Bridge over Tuttle Creek Reservoir, Riley County, Kansas
Owner: State Highway Commission of Kansas
Designer: Bridge Department, State Highway Commission of Kansas
Fabricator: Darby Corporation

Six steel bridges were chosen by this year's jury as the most beautiful opened to traffic during 1961.

Three Prize Bridges (formerly called "Award") and three Awards of Merit (formerly called "Honorable Mention") were selected from 78 entries in the AISC's annual bridge competition.

Prize Bridges will have newly designed stainless steel plaques affixed to them as a permanent tribute to their designers for combining aesthetics and utility in graceful river crossings. The designers, owners, and fabricators of the Prize Bridges and Awards of Merit will receive certificates, attesting to the selection of the bridges.

The final selection of the six bridges was unanimous, but was arrived at only after serious consideration of a number of other bridges.

Jury members were: H. Harvard Arnason, Vice President for Art Administration, The Solomon R. Guggenheim Museum, New York City; Charles R. Colbert, Dean, School of Architecture, Columbia University, New York City; Dr. G. Brooks Earnest, President, American Society of Civil Engineers, Fenn College, Cleveland, Ohio; Luther Lashmit, Lashmit, James, Brown & Pollock, Winston-Salem, North Carolina; and Edward W. Tanner, Tanner, Linscott & Associates, Inc., Kansas City, Missouri.

SHORT SPAN PRIZE BRIDGE WITH FIXED SPANS UNDER 400 FEET AND COSTING LESS THAN $500,000

Skykomish River Bridge, Sultan, Washington
Owner: Snohomish County, Washington
Designer: Harry R. Powell & Associates
Fabricator: United Concrete Pipe

SHORT SPAN BRIDGE, AWARD OF MERIT

Pecwan River Bridge, Weitchpec, California
Owner: State of California
Designer: State of California
Fabricator: American Bridge Division, United States Steel Corporation

SHORT SPAN BRIDGE, AWARD OF MERIT

South Pond Bridge, Lincoln Park, Chicago
Owner: Chicago Park District
Designer: Engineering Department, Chicago Park District
Fabricator: Vierling Steel Works

The Pacific Northwest Bell Telephone Company's Fourth Avenue Building in Portland, Oregon, is under construction as a three-story building, designed to have five more stories added at some future date.

FIRST USE OF NEW AISC SPEC

The first use of the new AISC **Specification for the Design, Fabrication and Erection of Structural Steel** for an entire building is the Pacific Northwest Bell Telephone Company's Fourth Avenue Building in Portland, Oregon.

John T. Merrifield, chief engineer for Moffatt, Nichol & Taylor, structural engineers, heartily praised the new Specification: "Once the designer is familiar with its provisions, it is no more difficult to work with than the old Spec. Most important, it allows substantial savings in steel tonnage – savings that can be passed on to the owner. It also means savings in the weight of the building, which, in certain cases, can mean reduction in foundation costs." Mr. Merrifield also indicated that the resulting structural frame will be easy to erect and is economically efficient.

The building is 200 x 200 ft, with a gross floor area of 120,000 square feet. It will ultimately be eight stories high, although initial construction is only three. Columns are spaced at 24 x 36 ft, with live loads at 100 and 150 psf.

Foundation conditions dictated a simple framing system with no moment connections. For this reason all lateral loads will be resisted by vertical concrete stair and elevator shafts with no lateral loads in the steel frame. Floor diaphragms will consist of cellular units with a two-and-one-half-inch concrete cover.

By mutual agreement with architects Stanton, Boles, Maguire and Church, and Mechanical Engineers J. Donald Kroeker & Associates, the framing was made sufficiently deep to allow passage of air ducts through the webs of all floor beams and girders. Allowance was made for a 12-in.-deep duct to pass through any member.

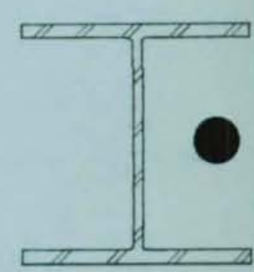
High strength steel was the answer to this situation, and the new AISC Specification (Nov. 1961) with new height-to-thickness ratios for plate girders was made to order. The 21½-in.-deep floor beams, spaced nine feet on centers and spanning up to 27 ft, are small plate girders of A441 steel, utilizing a one-quarter inch web plate.

The deeper beams have the advantage of greater stiffness, making it possible to design for stress without compensating for deflection, thus getting full work from each pound of steel used.

Girders are 27 and 30 WF sections of A440 and A441 steel. Many of the girders are the new lightweight sections recently added to the sections list. Columns are 12 and 14 WF sections of A440 and A441 steel.

Connectors, with the exception of a few special welded connections, are A325 high tensile bolts using the new bearing type connections which exclude bolt threads from the bearing surfaces.

Ross B. Hammond Company is the general contractor, and Bethlehem Steel Company is fabricating 613 tons of structural steel. All are Portland firms.

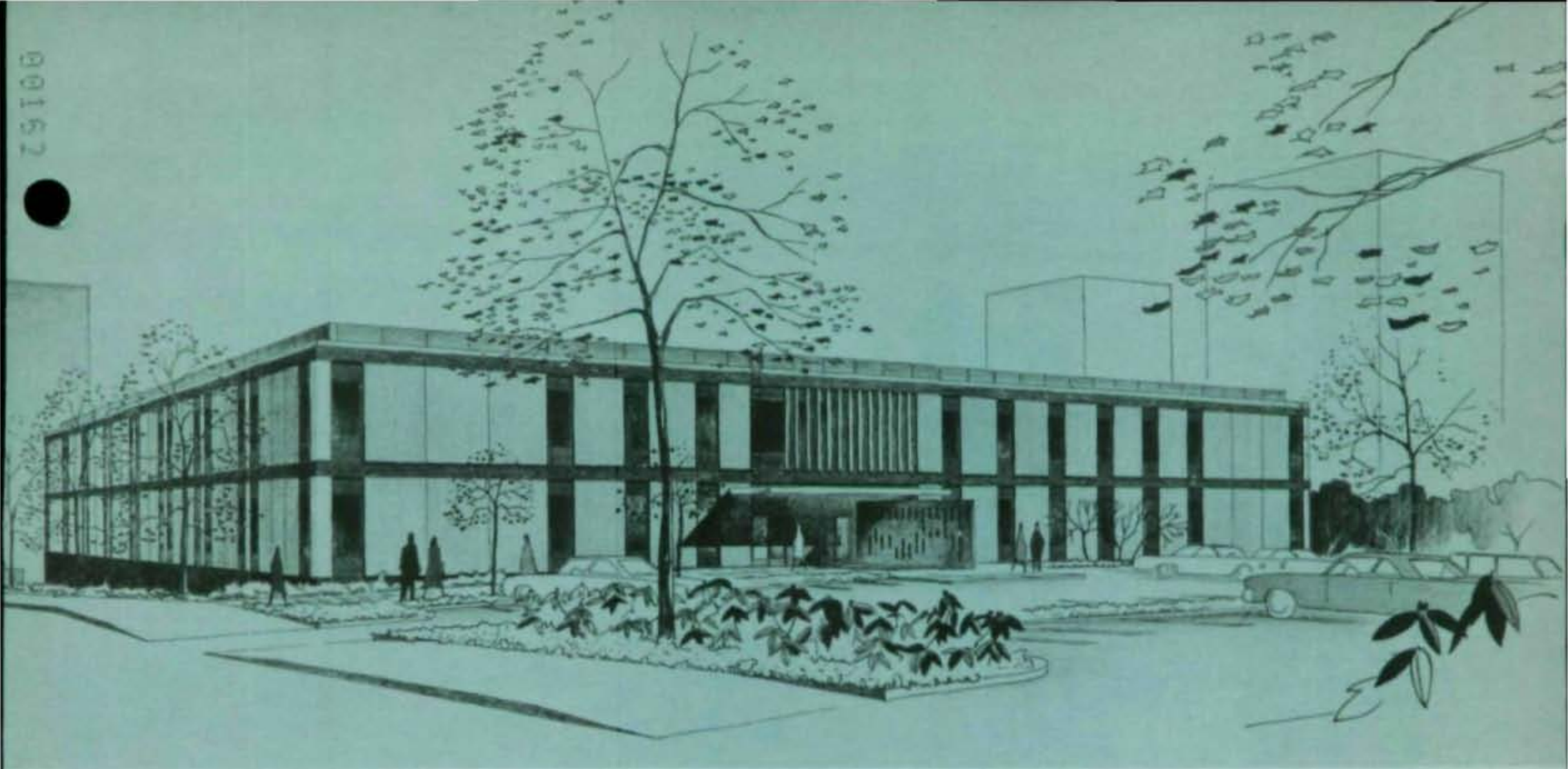


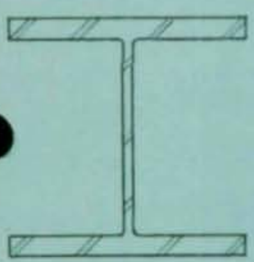
A440 Steel
14 WF 103

RELATIVE

A440 or A441 Steel
27 WF 94

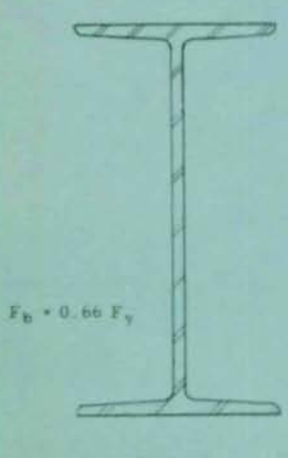
RELATIVE





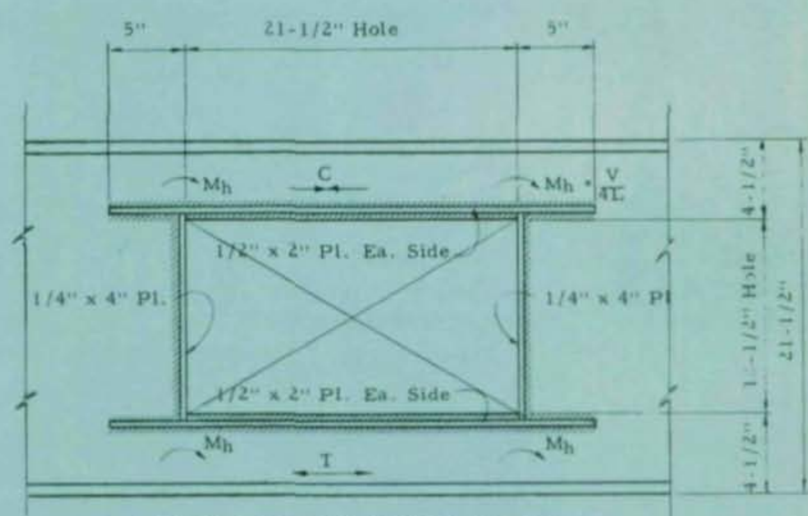
A36 Steel
14WF 136

COLUMN SIZES

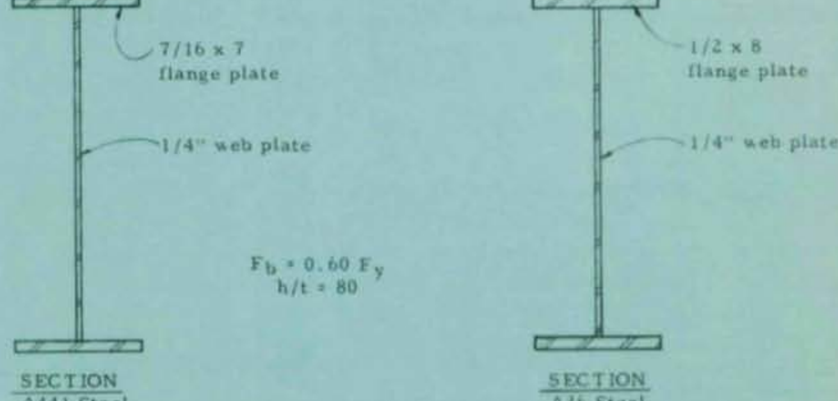


A36 Steel
27 WF 145

GIRDER SIZES



FLOOR BEAM DUCT HOLE



RELATIVE FLOOR BEAM SIZES

SIX LANES ADDED TO GEORGE WASHINGTON BRIDGE

When he designed the George Washington Bridge in 1930, Othmar H. Ammann with remarkable foresight provided for the future addition of a lower level, making allowances in the foundations, towers, cables, suspension system, and even providing steel connecting plates for its attachment.

All this preliminary planning paid off when the Port of New York Authority decided that traffic projections for the future dictated the need for the lower level addition now. The construction of the six additional lanes and expanded approaches on the New York and New Jersey sides of the Hudson River started in September 1958, and was completed in August 1961 without interruption to traffic on the upper level. The famous bridge now holds the record of the only 14-lane suspension bridge in the world.

The preassembled sections were raised from each shore or from barges anchored in the river by erection trolleys that were suspended from temporary tracks under the existing span. Each of the platforms was complete with power tools and other equipment. The sections were then moved to their permanent positions with cables attached to the hoisting engine and then fastened to the steel connecting plates.

Bethlehem Steel Company fabricated and erected the structural steel sections, the first time this type of construction has been performed.

Seventy-five of the sections weighed 220 tons, were 108 ft wide, 90 ft long and 30 ft deep; the 76th span weighed 62 tons and fitted the 60-ft gap at the center.

Vital statistics for the bridge are: It is the third longest suspension span in the world – 3500 ft.

Length of main span between anchorages – 4,760 ft.

Width of bridge – 119 ft.

Width of roadway – 90 ft.

Height of the tower above water – 600 ft.

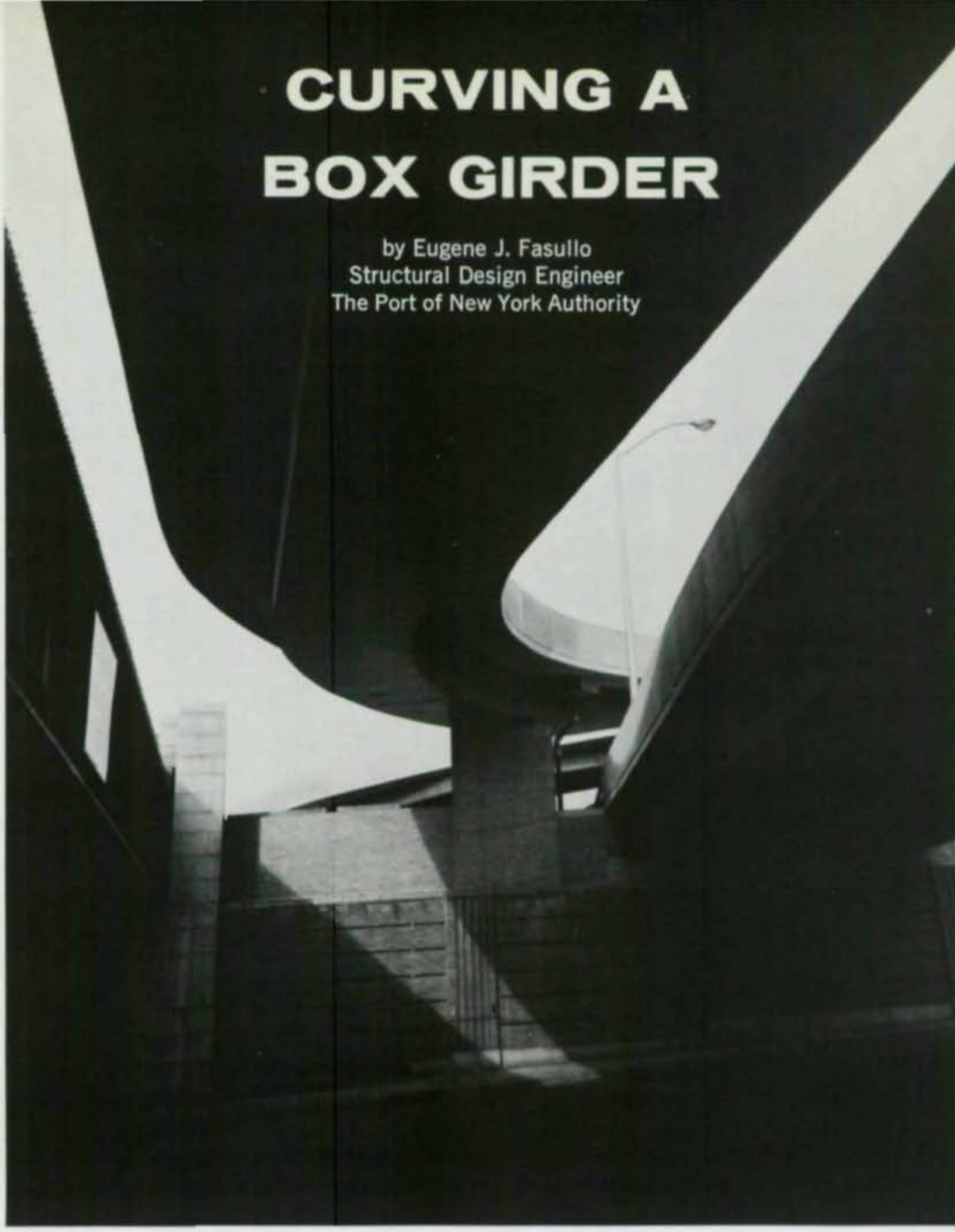
Channel clearance at midspan – 212 ft.

In 1961, eight lanes carried 37,998,600 vehicles.

Projected 1963 traffic for 14 lanes – 43,000,000 vehicles.

Original cost of the structure – $59,000,000;
with approaches – $76,200,000.

Lower level project and a new bus station cost $145,000,000, bringing the total investment to $221,200,000.



CURVING A BOX GIRDER

by Eugene J. Fasullo
Structural Design Engineer
The Port of New York Authority

Every day 182,000 people pass through the Port of New York Authority Bus Terminal in Manhattan, and the number is always increasing. Part of the expansion program to meet this demand is the three-level addition (**Steel Construction Digest,** Volume 17, Number 4) to the Terminal that will increase its capacity by 50 percent. Ramps will allow direct access to the bus and parking levels from the Lincoln Tunnel without using city streets.

Part of the ramp complex is a two-span continuous steel curved box girder. The horizontal alignment of the roadway carried by the box girder is composed of two sections: a straight section 58.5 ft long and a circular curved section with a 67-ft radius extending 125 ft beyond the point of curvature. The roadway is on a longitudinal grade of eight percent (see Figure 1). Several girder layouts were studied using angle points and introducing an additional supporting pier in the curved portion to make the girders conform to chords of the curve. All these studies were rejected because they resulted in large sidewalk projections beyond the curb line, some as long as nine feet. These excessive sidewalks represented wasted concrete and steel, as well as poor appearance and expensive deck slab formwork caused by variable slab overhang.

Calculations were then made to determine the feasability of utilizing a curved steel box girder which would follow the roadway alignment. It was concluded that a two-span continuous box section would be best suited to resist the flexural bending moments, shears and torsions which were present at every section along the bridge. This solution provided a more economical and pleasing design.

The first span is 110 ft-6 in. long, having a girder depth of 4 ft-6 in. The second span is 73 ft long and has a depth varying from 4 ft-6 in. to 2 ft.-9 inches. The change in depth was required to obtain the necessary clearance for a lower roadway. The girder itself is an all-welded box section eight feet wide composed of three-quarter-inch flange plates, three-eighth-inch web plates, and four 6 X 6 X ¾-in. corner angles. The top flange plate of the box girder conforms to the cross slope of the slab. The bottom plate is horizontal, normal to the roadway direction. In addition, five-inch tee sections are welded to the flange plates to avoid local buckling. Vertical stiffeners were spaced at 2 ft-6 in. and interior diaphragms at ten feet (see Figure 2).

The girder was analyzed by using the method of virtual work. Utilizing the three boundary conditions that the vertical displacement at M, torsional rotation at M, and the torsional rotation at O must be zero, we can write the three equations necessary to solve for the three unknown reactions.

The actual and primary structure with the appropriate boundary conditions is indicated in Figure 3. The actual structure is statically indeterminate to the third degree. For the analysis of this system, the displacements, slopes, and rotations of the actual and primary system must be determined (see Figure 4).

All notation is similar to that found in **Analysis of Statically Indeterminate Structures** by Parcel & Moorman. The internal work due to vertical shear is small and was neglected in this analysis.

With full torsional restraint provided at all three supports, it was found that the uplift on the inner bearing due to torsion at the two end-supports was larger than the downward vertical shear. This condition is difficult to accommodate at an expansion bearing. Therefore it was decided to remove the torsional restraint at the two end-supports and allow the girder to rotate under the total dead load condition and provide torsional restraint at these supports only for live load, which is only a small part of the total load.

This approach proved very successful, eliminating the uplift problem as well as reducing the torsional stresses, yet providing the desired rigidity under live load. A second analysis was carried out for the dead load condition, which was indeterminate to the first degree, and the moments, shears and torsions combined with the live load case for use in the final design. The moment, torsion, and shear diagrams are shown in Figure 5.

To accomplish the requirement of a dual boundary condition, three bearings were provided at each end support, Figure 6. The center bearing allows rotation about the longitudinal and transverse axes and the outer bearings allow rotation about the transverse axis

The curved box girder supports a roadway ramp to the new upper level of the Port of New York Authority Bus Terminal on Eighth Avenue in New York City.

Figure 1: Plan showing horizontal alignment of curved girder and location of supports.

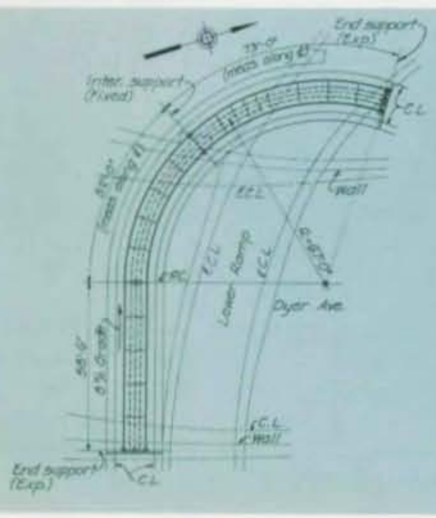


Figure 2: Typical cross section showing girder and roadway slab details.

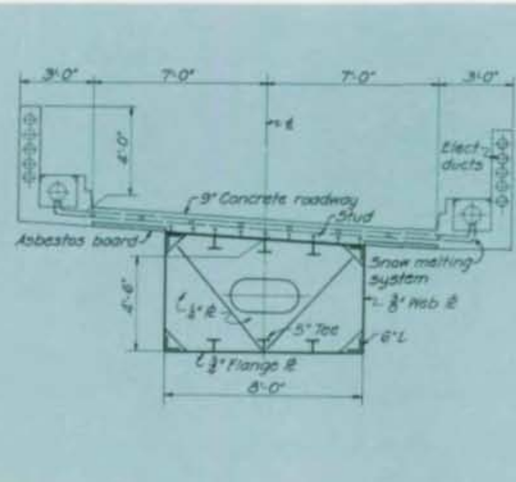
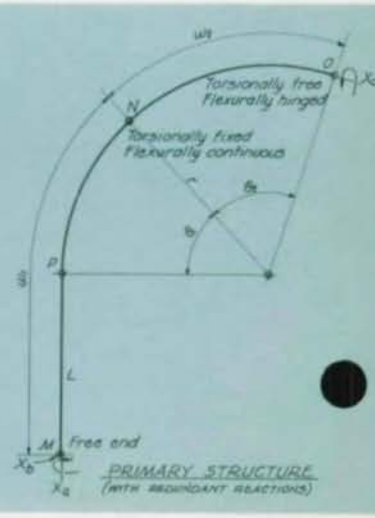


Figure 3: The actual and primary structure with the appropriate boundary conditions used for design.



only. The center bearing was utilized until the full dead load of the steel and concrete deck were in place, at which time the two outer bearings were placed and the center bearing removed. Temperature movement was provided for by allowing a longitudinal translation at the end-support bearing, which was radial to the interior support.

The curved steel box girder was fabricated in the shop in three sections. The diaphragms were originally designed as a K-brace type, but the fabricator elected to use solid-plate milled diaphragms with an access hole. This enabled him to control the cross-sectional shape along the girder more accurately.

Only two field splices were required and were located at points of low moment. High strength bolts were provided at the field splices which would safely support the girder weight so that a full field butt weld could be made after erection without the need for any falsework. To insure against erection problems, a full shop assembly was specified. The erection went smoothly and required the detouring of tunnel traffic below for only two hours. The field butt welds were fully checked by use of radiographic inspection.

Shear studs placed one foot on center were welded to the top flange to hold securely the 20-ft-wide concrete roadway which was poured in place. Because of heating elements imbedded in the concrete slab for snow melting purposes, the concrete slab was not counted on to help carry the live load composite with the steel box section. All the load was taken by the steel box section.

Formwork for pouring the roadway slab was hung from the box girder. The constant overhang of the slab from the box girder greatly simplified the construction of the formwork and placing of the reinforcing steel. A retarding agent was added to the concrete to insure that the steel box section would carry all the dead load as assumed in the design. Both the girder deflection and the girder rotation were measured during the various stages of construction and agreed very closely to the computed values. To increase the efficiency of the snow-melting system provided in the slab, the underside of the concrete was insulated with an asbestos board and the underside of the top flange plate of the box girder was sprayed with polyurthane foam.

In conclusion, it appears that the use of continuous curved steel bridges are economically feasible, structurally sound, and aesthetically pleasing. They are especially useful in the construction of intricate superstructures associated with complex highway interchanges. Sound engineering as well as competent fabrication and erection procedures are needed to assure success.

This design was conceived and developed by Arne Lier, structural engineer, and the author, of the Port of New York Authority Engineering Department, J. M. Kyle, chief engineer. Harris Structural Steel Corporation fabricated and erected the curved steel box girder.

$$\delta_a = 0 = \delta'_a + X_a\delta_{aa} + X_b\delta_{ab} + X_c\delta_{ac}$$
$$\delta_b = 0 = \delta'_b + X_a\delta_{ba} + X_b\delta_{bb} + X_c\delta_{bc}$$
$$\delta_c = 0 = \delta'_c + X_a\delta_{ca} + X_b\delta_{cb} + X_c\delta_{cc}$$

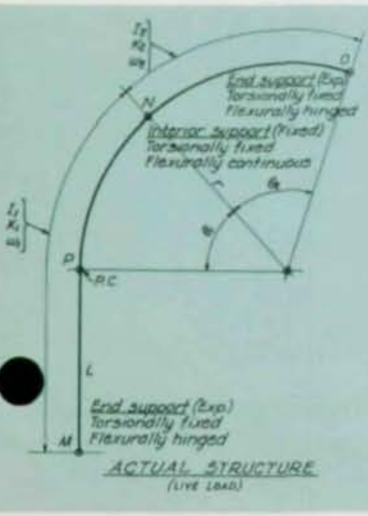
Where typically:

$$\delta'_a = \int_0^L \frac{Mm_a\,ds}{EI_1} + \int_0^L \frac{Tt_a\,ds}{GK_1} + \int_0^{\theta_1} \frac{Mm_a\,ds}{EI_1} + \int_0^{\theta_1} \frac{Tt_a\,ds}{GK_1} + \int_0^{\theta_2} \frac{Mm_a\,ds}{EI_2} + \int_0^{\theta_2} \frac{Tt_a\,ds}{GK_2}$$
$$\delta_{aa} = \int_0^L \frac{m_a m_a\,ds}{EI_1} + \int_0^L \frac{t_a t_a\,ds}{GK_1} + \int_0^{\theta_1} \frac{m_a m_a\,ds}{EI_1} + \int_0^{\theta_1} \frac{t_a t_a\,ds}{GK_1} + \int_0^{\theta_2} \frac{m_a m_a\,ds}{EI_2} + \int_0^{\theta_2} \frac{t_a t_a\,ds}{GK_2}$$
$$\delta_{ab} = \delta_{ba} = \int_0^L \frac{m_a m_b\,ds}{EI_1} + \int_0^L \frac{t_a t_b\,ds}{GK_1} + \int_0^{\theta_1} \frac{m_a m_b\,ds}{EI_1} + \int_0^{\theta_1} \frac{t_a t_b\,ds}{GK_1} + \int_0^{\theta_2} \frac{m_a m_b\,ds}{EI_2} + \int_0^{\theta_2} \frac{t_a t_b\,ds}{GK_2}$$

δ'_a – Torsional rotation at M due to the loads on the Primary Structure
δ_{aa} – Torsional rotation at M due to a unit torque at M
δ_{ab} – Torsional rotation at M due to a unit load at M
δ_{ac} – Torsional rotation at M due to a unit torque at O
X_a – Redundant torque at M
X_b – Redundant vertical reaction at M
X_c – Redundant torque at O

GIRDER ANALYSIS

Figure 4: These basic equations, which satisfy the boundary conditions, are used to solve for the three redundant reactions under the live load case.



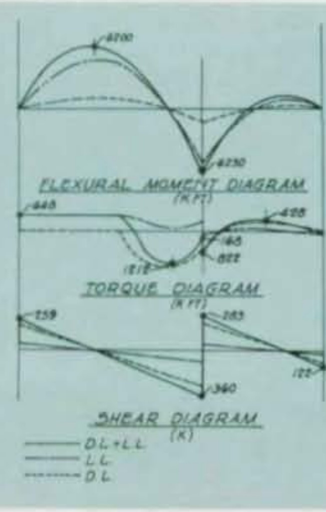
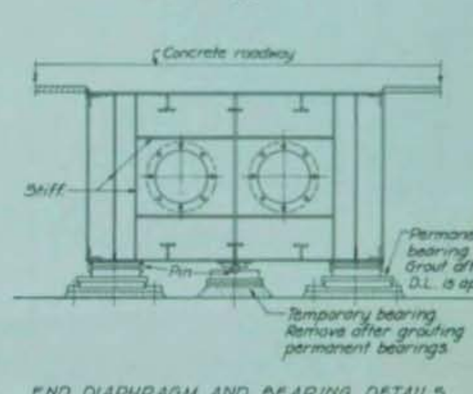


Figure 5: The flexural moment, torque and shear diagram for the dead load, live load and combined conditions.

Figure 6: Bearing details utilized to achieve a dual boundary condition at each end support.



THE CURVED STEEL GIRDER

It is a structural form previously avoided like the plague by bridge design engineers . . .

. . . but here's a significant pioneering effort produced by a co-ordination of sound structural design, fabrication and erection procedures.

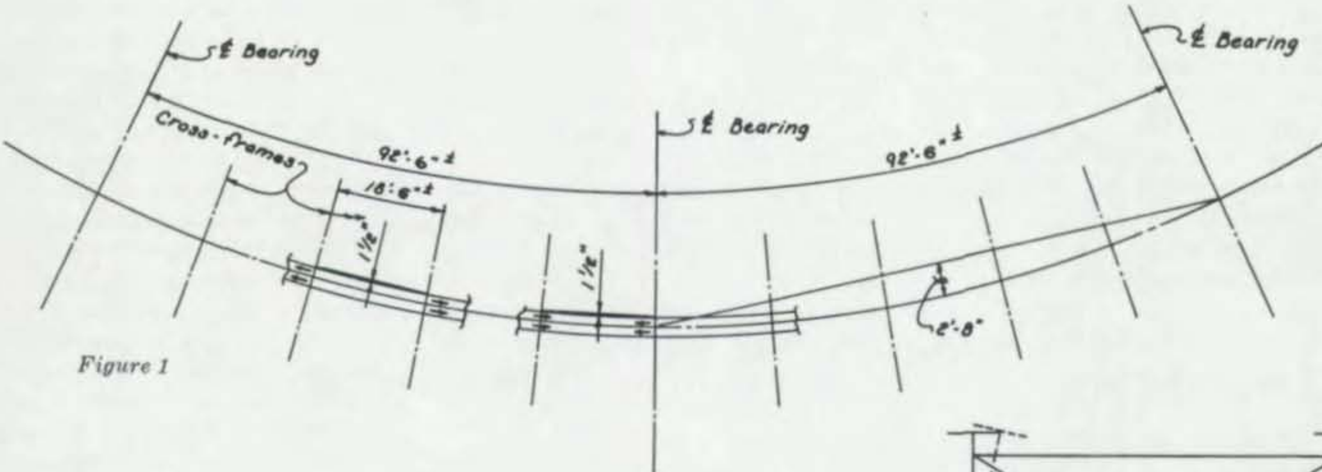


Figure 1

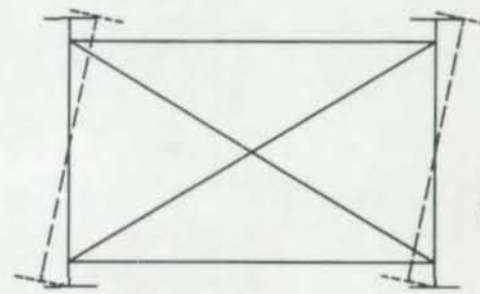


Figure 2

by Albert P. Bezzone, Jr.
Project Designer
and John J. Kozak, Chief Engineer
of the
California Division of Highways

Reprinted from **The American Road Builder**
The American Road Builders Association

A few years ago the bridge engineer investigated a stream, found a likely place for the bridge, and built it; all the road engineer had to do was get a road to it. The balance shifted, and now the road engineer builds a way to the stream; the bridge engineer has to build a bridge to accommodate it.

Modern highway design has made this a challenging problem indeed, and especially where the requirement is for curved alignment, long spans, restrictions on use of falsework and need for good appearance. This is the story of such a bridge — the first completely curved steel bridge ever constructed and made possible by the adaptability and versatility of welded steel construction. The bridge is part of the Golden Gate Pasadena Freeway Interchange in Los Angeles.

The requirements facing our bridge designers were to design a structure to bridge a large flood control channel, on sharply curved alignment with design span lengths of 92½ ft and curve radius of 400 ft, to make the structure pleasing in appearance because of its location in full view of heavy freeway traffic and to achieve economy along with structural integrity and aesthetic acceptance.

Solutions which had been used successfully in solving previous bridge problems were studied carefully and adaptations attempted to suit the controls listed above. Because of the curved alignment and limitations on the use of falsework, it was soon clear that some form of steel structure would have to be used.

Several girder layouts were studied using angle points to make the girders conform to chords of the curve. The angle point layout was rejected because of its poor appearance, expensive deck slab framework caused by the variable slab overhangs, and expensive expansion details (if expansion is provided at an angle point).

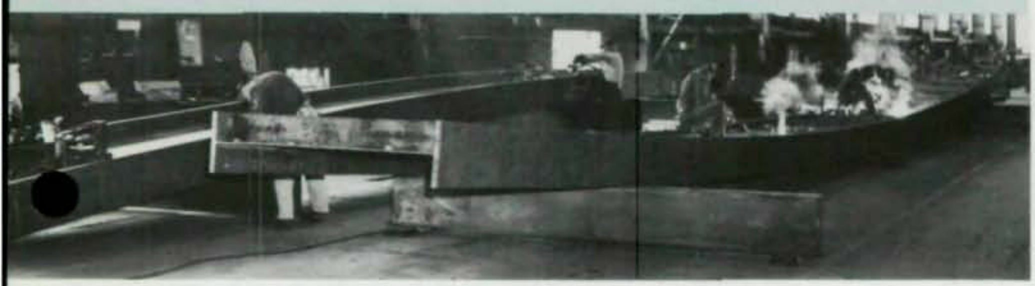
Preliminary calculations then were made of the forces and stresses resulting from completely curved construction. It was obvious that individual curved girders were unstable but that pairs of girders which were effectively braced would increase torsional stiffness considerably. Preliminary investigation of fabrication and erection problems also revealed that curved construction was feasible.

The design of welded girders made up of three plates connected by continuous fillet welds is a well-established and routine technique. It is necessary, therefore, to discuss only those features which are unique in curved structures.

Curvature causes an eccentric effect in the flanges between points of lateral support. This effect results in lateral bending, adding to the stress in the flange on one side of the web. The distance between cross-frames is 18½ ft. Figure 1 shows the effect of eccentricity on the tension flange in the area of maximum positive moment.

A similar calculation holds for the compression flange under dead load. For live load the compression flange has continuous lateral support from the composite concrete deck.

Photographs show the curved steel girders being fabricated in the shop (below) and erected ready for the placement of the composite deck.



Another effect of curvature concerns the deformation caused by torsional stresses in the flanges. At the center support, the torsion causes a rotational movement toward the center of the curve. In the area of positive moment the rotation is away from the curve center. Opposite to this movement is the tendency of the girder to twist because of deformations in the flanges due to longitudinal bending. In this case the compression flange shortens and moves laterally toward the center of the curve. The tension flange, conversely, moves away from the center and the girder tries to twist. See Figure 2.

The bracing system is important in any structure and is of special importance in a curved steel bridge. Bracing must be designed to carry lateral forces from wind, centrifugal effect, seismic forces when they control, and torsional forces along the structure and into the end bearings. The end cross-frames should be heavy enough to provide an ample factor of safety.

Where a curved structure such as this one is used, the cross-frames combine with the bottom lateral bracing to take the lateral loads due to dead load. For live loading the composite concrete deck provides a stiff lateral diaphragm to work with the bottom lateral bracing.

In order to increase the stability of the top flange during placing of the deck slab concrete, a sequence of deck construction was provided. This sequence resulted in the center section of slab in each span being placed after the end sections had gained enough strength to act as effective horizontal diaphragms. A full top lateral system would accomplish the same purpose but would be an added expense and would interfere with formwork for the top slab.

End bearings were designed to carry uplift which could occur with critical combinations of lateral loading and torsional effects. These bearings allow rotation but no translation. Longitudinal movement for this structure will be provided by movement of two tall flexible bents. If it had been necessary to allow longitudinal movement at the end bearings, these bearings could have been designed to allow the movement and still prevent vertical movement.

The girders were fabricated in 30-ft segments by cutting the flange plates on the prescribed curve from straight plates. Web plates were thin enough to conform easily to the curve, and the plates were held in a modified jig.

Continuous fillet welds at the junction of web and flange were made by submerged arc machine welding. The machine followed the curves with no difficulty. There were no unbalanced residual stresses, and the girders held required shape when loaded.

Although comparative costs are not available it can be concluded that there were no unusual complications in the fabrication of the curved girders over straight construction.

Girders were assembled in pairs in the shop with bracing securely installed. Fit of the field splice locations was checked in the shop before shipment. The 12 percent super-elevation combined with the horizontal curvature made the shop fitting especially important. Field splices were located at the 0.2 point of the span from the center support.

The assembled braced pairs of girders were erected by two truck cranes. These sections were connected temporarily at the field splice by clip angles and bolts. Cross-frames and bottom lateral bracing were then installed in the center bay and the field splices were welded. Thorough preliminary fitting by the fabricator resulted in smooth erection procedures.

We believe that experience with curved steel girder design and construction will open new horizons for the use of welded steel. Sound engineering as well as competent fabrication and erection are needed to assure success. It is hoped that other bridge engineers will be encouraged to use functional, economical, and attractive welded curved steel construction.

Kaiser Steel Corporation fabricated and erected the curved steel girders; general contractors were the Vinnell Corporation, Vinnell Constructors and A. S. Vinnell Company.

The authors were responsible for the design under the general supervision of James E. McMahon, California's assistant state highway engineer – bridges.

QUESTIONS & ANSWERS on the AISC Spec

Q. What is proper value of unbraced length of the compression flange in the conditions shown in sketches 1, 2, 3, and 4 when interpreting Section 1.5.1.4.5 of the new AISC Spec.?

A. The Boston Society of Civil Engineers **Journal** of October 1954 contains a table showing theoretical values for $k\,l\,d/bt$ based on a review of available theory on buckling phenomena. This indicates that the value of k for cantilever beams should be less than 1.0, and in all cases less than that for a simple beam of the same span. This is based on the fact that a cantilever beam always has lateral support of the compression flange at its point of maximum moment, while a simple beam must have its compression flange braced where moment is high, and the susceptibility to torsional buckling is greatest. In other words, using k of 0.85 for a cantilever beam unsupported against either translation or rotation, as in sketches 1 and 2, would be on the conservative side (since k of 0.81 is shown for the most severe case of a concentrated load applied on the top flange). For any case of a cantilever loaded less severely than this, such as in sketches 3 and 4, a value of $k=1.0$ for $k\,l$ for the unsupported length of compression flange is on the conservative side.

Q. Why are all conditions shown in sketches 1, 2, 3, and 4 considered equivalent?

A. See answer to question 1. They are not equivalent, but a value of $1.0\,l$ is on the conservative side for even the most critical case, as shown in sketch 1. For sketches 2, 3, and 4 it is even more conservative, but no refinements were made. All these cases are more conservative than that using $1.0\,l$ for unsupported length of the compression flange for a simple beam.

Q. If either the distance l_1 or l_2 in sketch 5 is greater than 13b, is it necessary to provide a brace to qualify for $F_b = 0.66F_y$ (assuming a compact section)?

A. If distance l_1 is greater than 13b, the bottom flange must be braced at a distance 13b from the column line. If distance l_2 is greater than 13b, top flange bracing would be required between inflection points. Normally, roof or floor construction would be adequate to provide support to the top flange.

Q. In other words, is it always necessary to provide a brace at the point of contraflexure?

A. No – see question 3 above. Point of inflection may be considered to render "indifferent" but adequate lateral support.

Q. In sketch 6 would 2x be used as unsupported length? (Top flange is braced at center line).

A. No – "x" should be used as the unsupported length of top flange.

CANTILEVER & CONTINUOUS BEAMS

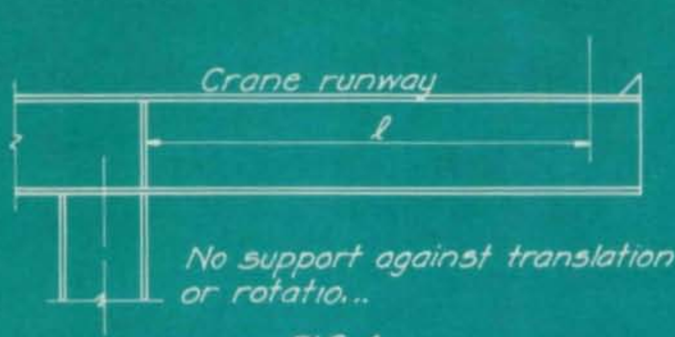


FIG. 1

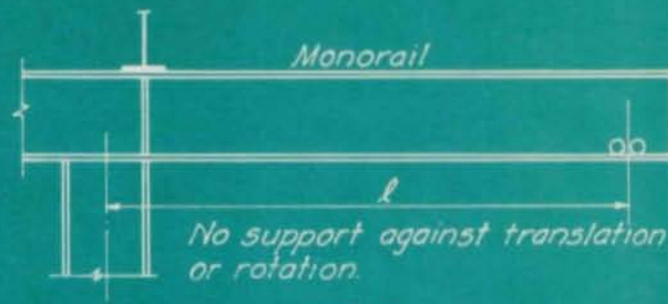


FIG. 2

FIG. 3

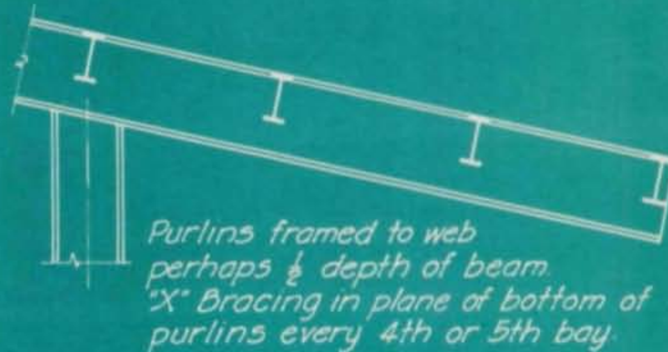
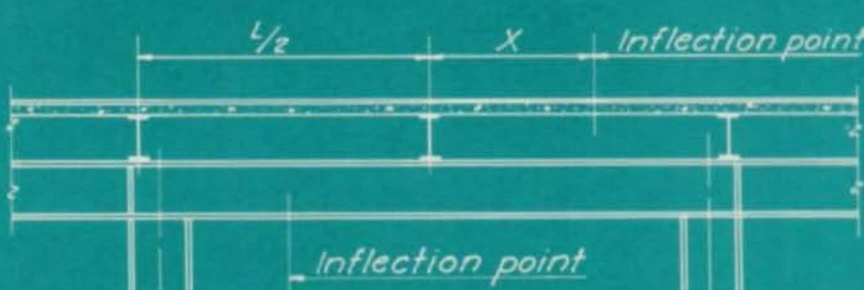


FIG. 4

Continuous beam - Compact section

FIG. 5



Continuous beam

FIG. 6

THE BATTLE OF INNER SPACE:

STEEL WINS

Left: The library's two reading rooms (one in foreground, the other in background at right) are separated by a low-ceilinged administrative section at left. Right: Dark areas of the library's exterior are steel painted dark green to contrast with the aluminum frames and to complement the grayish precast concrete panels.

Originally, the architects for the Columbia Avenue Branch of Philadelphia's Free Library conceived the building in concrete — but then the structural engineer came up with the preliminary framing sizes necessary. They soon decided that the concrete consumed far too much space. Solution of the problem: strong, inherently compact steel. The final design for the library demonstrates, once again, that only lightweight structural steel framing provides maximum usable space.

The building also handsomely expresses the most up-to-date architectural philosophy: frills and camouflage are out; frank baring of good framing is in. The structural steel is exposed not only on the exterior, but also on the interior, where roof framing forms an attractive and interesting grid pattern. The roof structure is four feet deep and consists of ten Vierendeel trusses 49 ft long, spaced 15 ft on centers, with longitudinal beams welded to the top and bottom chords of the trusses at third points.

Wright, Andrade, Amenta & Gage were the architects; Seymour Greenberg, structural engineer; P. Agnes, general contractor. The 55 tons of structural steel (A36) were fabricated by American Iron Works. All are of Philadelphia.