DEPARTMENT OF DESIGN

Problems in Fabrication and Erection of High-Strength Steels

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This paper discusses the types of bridge structures on which high-strength steels have been used and outlines the types of steels suitable for high-strength purposes. The main differences in fabrication between these steels and ordinary carbon steel are covered and also the differences in erection. Much is made of processes to be observed in design to assure that proper detailing of high-strength steel members is accomplished.

• USE of high-strength structural steels in bridge construction has been growing quite rapidly in recent years due to the fact that spans have been getting longer and longer, particularly in the case of highway bridges. Engineers have been forced to adopt long spans at many bridge locations for physical reasons at the site, as well as to meet architectural requirements. Traffic must be kept moving in a smooth flow, without bottlenecks, regardless of the difficulties involved in locating and constructing bridge structures suitable to carry the vehicles involved.

It is in the long spans that engineers have found high-strength steels to be economical, as the saving in dead weight and the reduction in foundation costs have more than balanced the higher costs per pound of the stronger steels. Some of the largest spans would, in fact, be almost impossible to build without the use of high-strength steel, as bridge members designed with ordinary carbon steel would be too large and heavy for practical fabrication and erection. The new road building programs now getting under way will undoubtedly increase the demand for stronger steels in highway bridges.

The following illustrate some recent bridges which have utilized high-strength steel to advantage.

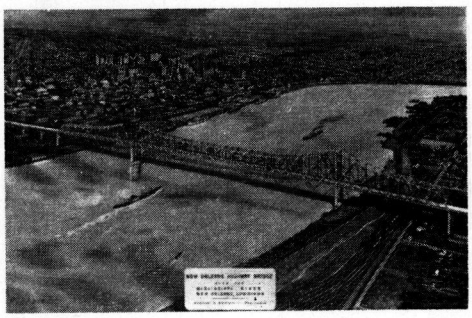

Figure 1. New Orleans Bridge over the Mississippi River.

Figure 1 is an architectural drawing showing New Orleans Bridge over the Mississippi River, now under construction, designed by Modjeski and Masters. The New Orleans anchor span is 853 ft long, the main span is 1,575 ft long, and the Algiers anchor span is 591 ft long. The suspended portion of the main span is 689 ft. The roadway is 52 ft wide, supported on rolled stringers and plate girder floorbeams. The majority of the truss members and the floorbeams are made of high-strength steel. This bridge will be the longest cantilever bridge in the United States.

Figure 2 shows the New Jersey Turnpike Bridge over Newark Bay, designed by Howard, Needles, Tammen and Bergendoff. It is a three-span continuous structure, the 670-ft center span being a tied arch. The side spans are each 300 ft long. Two 36-ft roadways are supported on rolled stringers and plate girder floorbeams. Most of the truss members are made of high-strength steel. Approach girders, on both sides, are shown (Figure 3) in the unloading yard. These

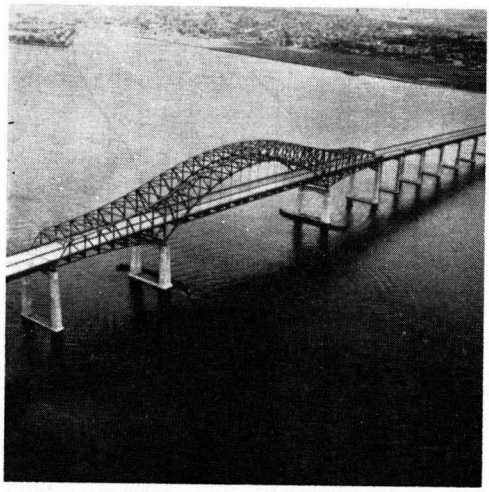

Figure 2. New Jersey Turnpike Bridge over Newark Bay.

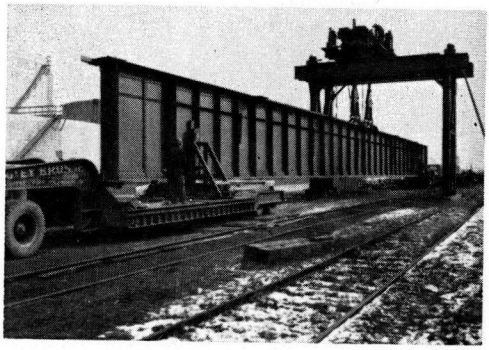

Figure 3. Approach girders for New Jersey Turnpike Bridge over Newark Bay.

girders are 170 ft long, 10 ft deep, and weigh about 106 tons each. Flanges consist of 12-in. side plates, 8- by 8-in. angles, and 25-in. cover plates. All of the main material in the girders is high-strength steel.

SPECIFYING AND DETAILING

For one reason or another considerable confusion exists among engineers and fabricators regarding this important subject. Some engineers do not know what type of steel to specify and some fabricators do not know how to process the various grades of high-strength steels. One of the purposes of this paper is to try to clear up some of the uncertainties in the minds of users.

Some of the high-strength steels are classified as "carbon steel" and others as "low alloy steel." Among the high-strength carbon steels, the most commonly used for bridges are ASTM A94 (silicon steel); Man-ten, produced by U. S. Steel; and medium manganese produced by Bethlehem; and similar steels produced by other steel companies. Among the low alloy steels, the most commonly used for bridges are Cor-ten, Tri-ten, and Tri-ten E produced by U. S. Steel; Mayari R and manganese vanadium produced by Bethlehem; and similar steels produced by other companies.

All of the steels mentioned in both groups, except A94 (silicon steel), and within the thickness limits to which they are produced, comply with ASTM-A242 (low alloy steel) specifications for tensile properties and bending properties, but they are not all in compliance with the requirements of A242 for weldability and corrosion resistance. In other words, they all have to meet the same requirements for strength and ductility. It is to be noted, however, that all of these steels have less ductility than ASTM A7 steel. To obtain high strength producers have had to sacrifice some of the ductility which is one of the most valuable properties of A7 carbon steel. Designing engineers recognize the loss of ductility in these steels by requiring a higher grade of shop workmanship when they are used.

It is customary for designers of long-span bridges specifying high-strength steel for highly stressed members to stip-

ulate that all rivet holes be subpunched and reamed, or drilled full size, regardless of the thickness of material. This requirement is made in order to avoid the possibility of getting incipient cracks around the edges of the holes, as such cracks might act as stress raisers promoting fatigue failures or brittle fracture.

Another requirement, essential for the high-strength carbon steels, but frequently specified also for the low alloy steels and even for A7 steel, refers to the treatment of sheared or flame-cut edges. The usual requirement is that sheared edges or hand-flame-cut edges shall be planed to a depth of ¼ in. to remove any incipient cracks. Machine-flame-cut edges may be used without removal of metal if they are softened by post-heating. Some engineers have also specified that a slightly greater edge distance be used for rivets for high-strength steel than for A7 carbon steel.

Figure 4 shows tension specimens of ¾-in. material, all of which met specification requirements for strength. However they did not all have proper ductility. Bar PH, which had flame-cut post-heated edges, fully met requirements for percentage of elongation and showed good reduction of area at the break. Bar SH, which had sheared edges, did not meet requirements for elongation and showed poor reduction of area at the break. Bar PL, which had planed edges, showed the same good results as bar PH. Bar AC, which had flame-cut edges with no post-heating, showed the same poor results as bar SH.

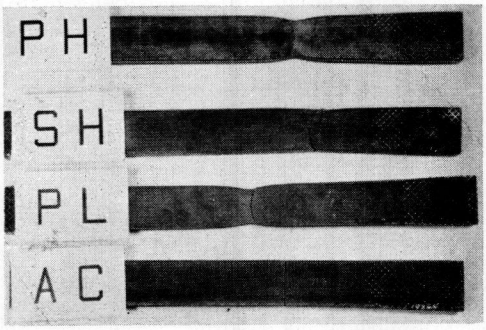

Figure 4. Tension specimens of 3/4-in. materials.

The same testing procedures were used for 9/16-in. material. The results shown (Figure 5) were practically the same as for the ¾-in. material.

Figure 6 shows bending tests of 9/16-in. material, in the top row of specimens, and ¾-in. material in the bottom row. Here, again, the bars with flame-cut post-heated edges and the bars with planed edges gave satisfactory results. The bars with sheared edges and flame-cut edges with no post-treatment gave unsatisfactory results.

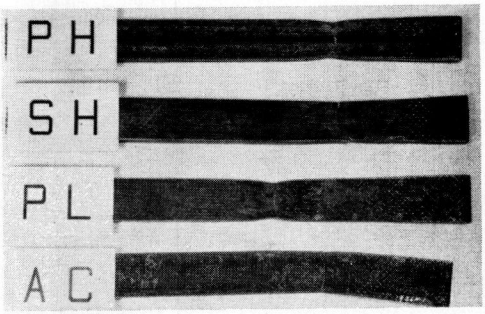

Figure 5. Tension specimens of 9/16-in. materials.

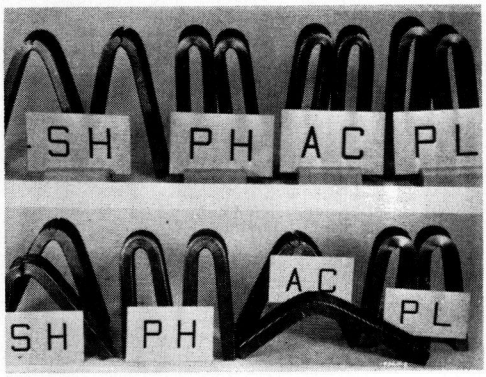

Figure 6. Bending tests of 9/16-in. specimens (top) and 3/4-in. specimens (bottom).

It may be interesting to know how the post-heating is done. Figure 7 shows a straight cut being made with a guided flame-cut. The post-heating torch is set 2 in. back of the cutting torch and is offset ½ in. inside the edge. The size of tips and the pressures vary with the thickness of material. Figure 8 shows a

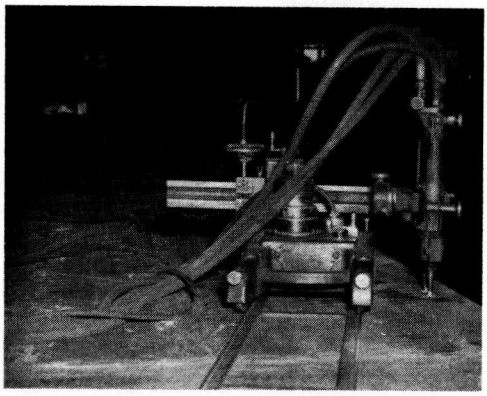

Figure 7. Post-heating torch following flame-cut torch to relieve edge stresses in plate.

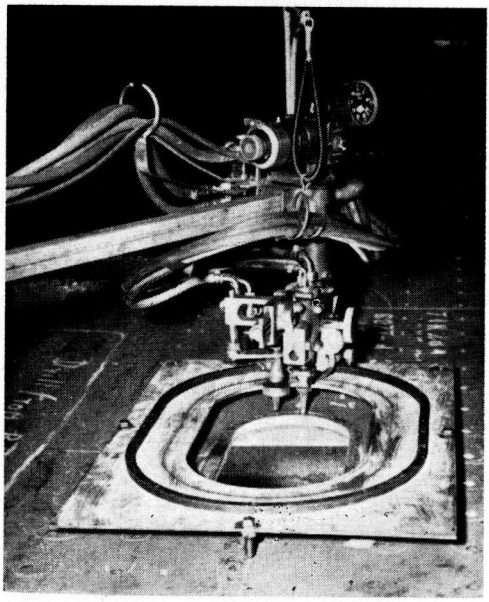

Figure 8. Curved flame-cut followed by post-heating torch.

curved cut being made for a perforated cover plate.

The high-strength steels included in the carbon group, although well-suited for use in riveted or bolted construction, are not too satisfactory for welded construction. The high strength is obtained by a high combined carbon and manganese content, which reduces the weldability of the steel.

It has been mentioned that all of the high-strength steels, except silicon steel, have the same strength and the same ductility. Silicon steel has a lower yield point and less ductility than the others. It has been used extensively for bridges in the past, but has fallen into disfavor with both producers and users in recent years for causes mainly due to the high carbon content permitted by the specifications. The majority of engineers now specify or permit one of the other high-strength steels to be used instead of silicon steel, because the yield point of silicon steel is lower than that of the others for all material thickness up to 1½ inches.

A242 specifications for low alloy steel are rather broad as far as chemical content is concerned and, therefore, permit many variations. Usually, however, when an engineer specifies A242 steel he wants a weldable high-strength steel with a high degree of corrosion resistance. It is, therefore, necessary for the producer to include such elements as chromium, copper, and nickel, even though they are not actually called for in the chemical composition. Such steels at Tri-ten E and manganese vanadium do not contain all of these corrosion resisting elements and, therefore, are not as corrosion resistant as engineers ordinarily require when they specify A242 steel. However, they are readily weldable and, of course, have equally high strength. They are quite similar in most respects to U.S. Navy high-tensile grade steel. The high-strength steels included in this low alloy group are suitable for welded, riveted, or bolted bridge construction.

The use of tack welding to aid in fitting up riveted work has become prevalent in fabricating shops. Caution must be observed, however, in tack welding high-strength steels in the carbon group. Tacking should be done by experienced welders, rather than fitters, using low-hydrogen electrodes in order to avoid the possibility of under-bead cracking. It is questionable whether tack welds should be permitted at locations which will be highly stressed in tension in the finished structure.

Because of its stiffness, high-strength

steel is somewhat difficult to draw up into perfect contact for riveting. It is advisable to use no rivets less than 1 inch in diameter. When sub-punched or sub-drilled 13/16 in., this permits use of ¾-in. fitting-up bolts, which are the smallest that will do effective work.

It should be recognized that the harder steels may require different tools for fabricating. Shears, punches, drills, and cutting edges must be stronger for high-strength steels than for carbon steels. Otherwise, more time must be allotted for the operations involved and allowance made for a greater breakage of tools. It might be of interest to note here, however, that the new type high-speed milling machines, which employ carbide cutters, will function just as well for high-strength steel as they do for carbon steel. When cold forming high-strength steels it is necessary to use a greater inside radius at bends than is customary with ordinary carbon steel.

An obvious item that has to be handled carefully in structural shops is the correct marking of different steels to make sure that they are fabricated with proper procedures and used in the right places. High-strength steel looks just like ordinary steel, so shop workers can easily become confused unless the identification is certain. The mills use a system of marking the ends of different kinds of steel with various colors of paint and this marking must be preserved during fabrication until the completed member receives its permanent mark.

The yield point of all high-strength steels, except A94 silicon, varies as the thickness varies over ¾ inch. This necessitates use of different unit stresses for different thicknesses. Fabricators have to detail their connections in accordance with the varying unit stresses used in the design of main members.

It is well known, of course, that high-strength steels have the same modulus of elasticity as ordinary carbon steel. This means that the high-strength steels have no more resistance to deflection and buckling than normal steels. As the stresses are usually higher, this fact forces designers, fabricators and erectors to take precautions to avoid excessive deflections and to prevent overall or local buckling of compression members.

Permissible ratios for plate width to thickness and plate depth to thickness are more severe for high-strength steels than for normal steels. Slenderness ratios

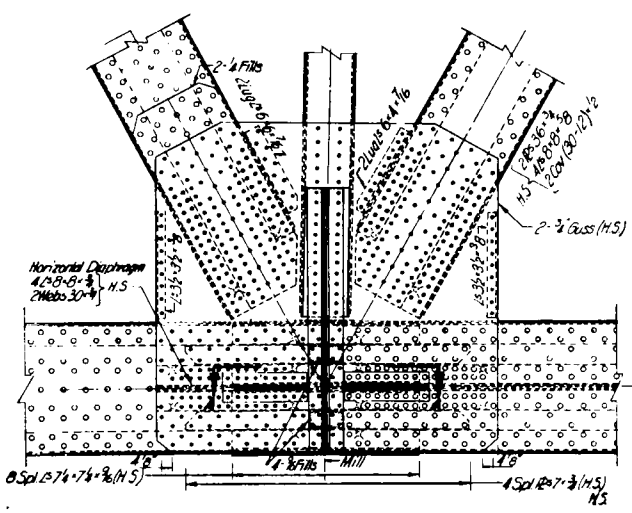

Figure 9. Truss joint of New Orleans Cantilever Bridge, showing stiffening angles along unsupported edges of gusset plate.

are also more severe, and requirements for stiffeners more stringent. Unsupported edges of gusset plates may have to be stiffened. All of these extra requirements for high-strength steels have to be observed in making shop details, in order to avoid the possibility of local buckling failures.

Figure 9 shows a well-designed truss joint in the New Orleans cantilever bridge. This shows stiffening angles along the unsupported edges of the gusset. The bottom chord consists of four web plates 48 in. wide and eight 8 x 8 angles, plus 30-in. horizontal web plates and perforated cover plates; it is spliced at the panel point. The 8-in. angles provide ample room for splicing with two lines of 1-in. rivets. Gage lines and edge distances can be spaced to suit specification requirements and to permit good riveting.

When only one line of riveting is contemplated the angles used should be 5 x 5, not 6 x 4 or 4 x 4. The upper sketches of Figure 10 show 5 x 5 angles, which provide sufficient clearance for driving 1-in. rivets and also permit proper gage and edge distance to be used for the splice angles or bars. The lower sketches show 6 x 4 angles, which do not permit splicing

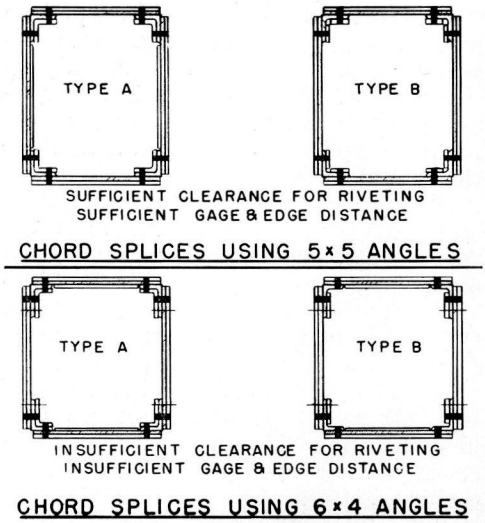

Figure 10. Comparison of chord splices using various angles.

to satisfy all requirements. This 4-in. leg angle encourages poor rivets and forces the detailer to cheat on edge distances, a comment that applies to A7 carbon steel as well as high-strength steel.

ERECTION PROBLEMS

In the erection of structures employing

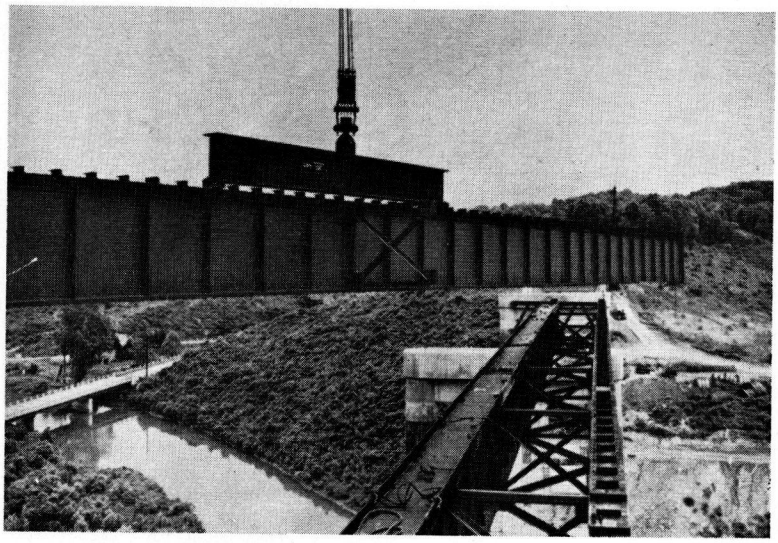

Figure 11. Girder braced with beam bolted flat. Succeeding girders held by falls until attached by cross-frames to stiffened girder.

high-strength steels, the principal problem is the one of guarding against an over-all buckling of main members. The unit buckling strength of a long slender member of high-strength steel is no more than that of a similar length carbon member and the area of the piece is ordinarily smaller. The compression flange of a long plate girder is particularly vulnerable to lateral buckling, especially when the girder is designed to act compositely with the concrete deck of the bridge. The steel compression flange of a composite girder usually has a low stiffness and therefore buckles easily if not supported properly. This type of long girder span has become quite popular in recent years; it therefore represents a continuing problem to steel erectors.

It is often necessary to reinforce the top flanges of long girders with temporary bracing of some kind for loading and erecting. The necessity for bracing has to be determined in the office before the girders are fabricated. Calculations in Bethlehem's office are based on the failure formula developed by de Vries (*1*). Safety factors based on these failure formulas vary with conditions, but average about 1.60. The temporary bracing may consist only of a deep beam bolted in flat position to the girder flange, or it may be a rather complex trussing system. Two adjacent girders may be permanently connected together before raising to brace each other, but this method doubles the weight of the lifted load, which might be objectionable from the equipment standpoint.

Figure 11 shows a girder with a beam bolted in flat position to stabilize the top flange. Other girders in the span after landing are held in the falls until they are attached by cross-frames to the stiffened girder.

Figure 12 shows a girder flange supported sidewise by a horizontal stiffening truss. Figure 13 is another view of the same girder, showing the truss and braces for supporting it vertically.

Figure 14 shows a braced pair of girders being placed on piers. The cross-frames are permanent, but the top flange bracing is temporary. Figure 15 is another view of the same span, showing an unsupported girder being placed along-

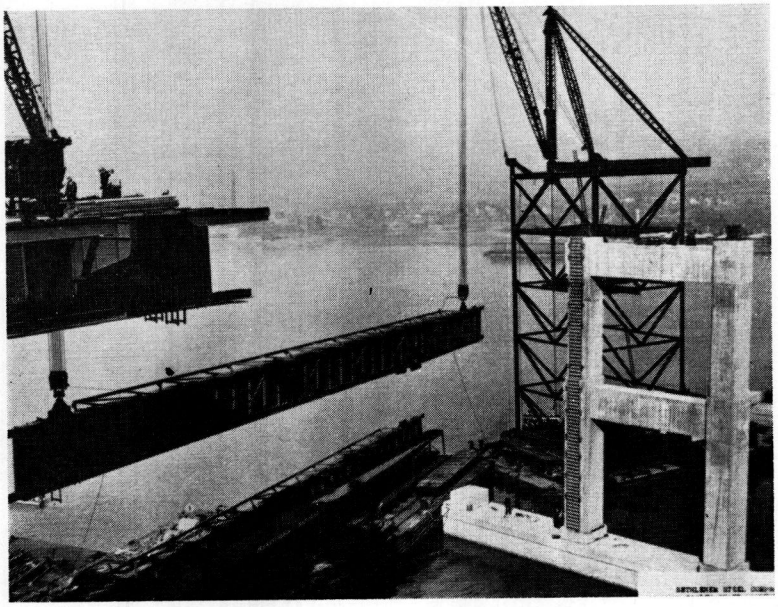

Figure 12. Upper flange of girder stiffened during erection with horizontal truss.

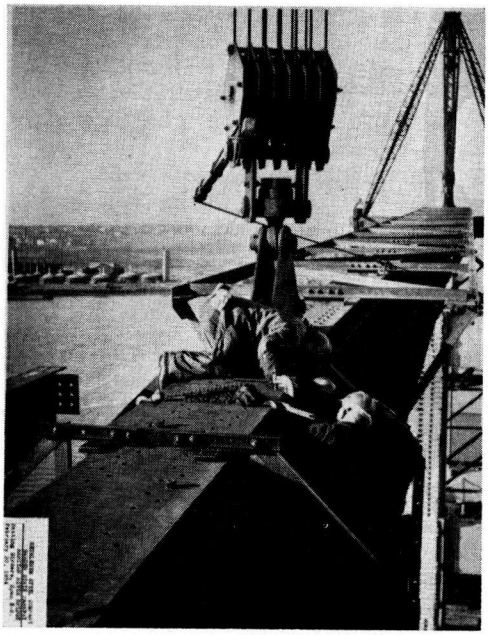

Figure 13. Close-up of girder of Figure 12, showing truss and braces for supporting it vertically.

side. After the cross-frames are attached the falls will be released.

When erecting a truss span by cantilevering it is necessary to know, ahead of actual erection, just how much the steel is going to deflect at each stage, as well as the shape of the erected portion. Of course, it is also necessary to know the stresses at all times. It is customary with high-strength steels to allow greater erection stresses than with ordinary steel; therefore, deflections and truss deformations are greater. More elaborate devices are, therefore, required to place the structure in its final position. Figures 16 and 17 show typical cantilever erection of a truss span and a girder span, respectively. The high unit stresses permitted with high-strength steels naturally produce greater movements during erection. Figure 16 happens to be a simple truss span, but it is being erected by the cantilever method to avoid an excessive amount of falsework. Trusses of high-strength steel can be cantilevered for longer distances than carbon steel.

The impression probably has been

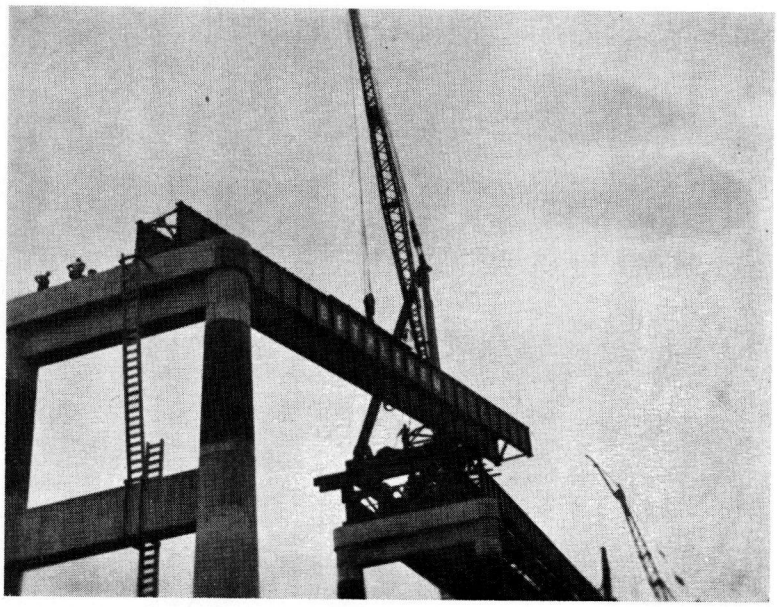

Figure 14. Braced pair of girders using permanent cross-frames and temporary top flange bracing.

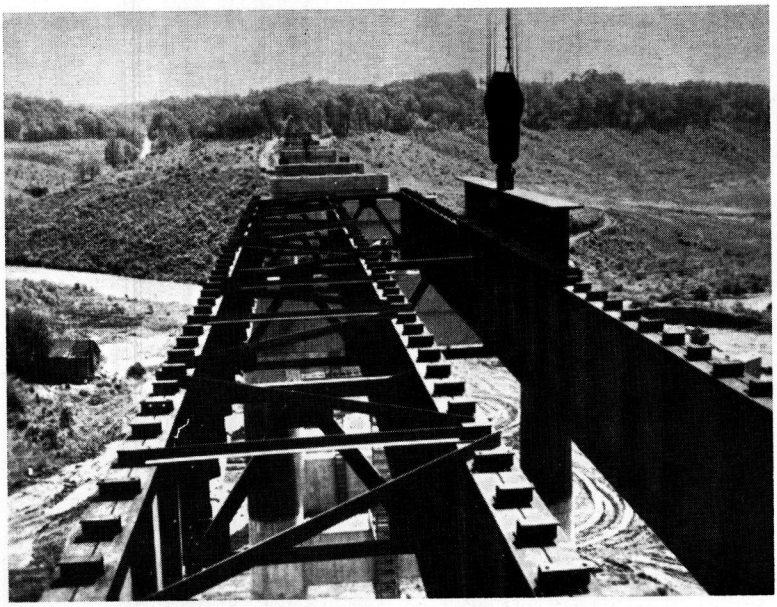

Figure 15. Third girder being placed alongside braced pair.

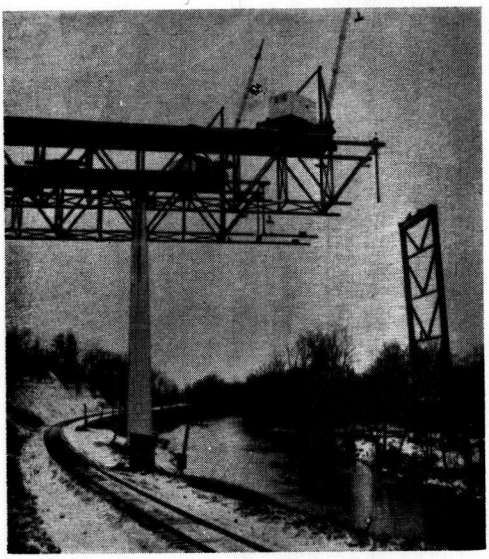

Figure 16. Typical cantilever erection of truss span to avoid excessive falsework.

gained from this discussion that the fabrication of a high-strength steel member is more expensive than a carbon steel member of similar make-up and dimensions. For most of the high-strength steels the increase in fabricated cost is about 5 percent. The increase for silicon steel is about 10 percent. These percentages refer to lump sum costs, not costs per pound. The percentage increases in costs per pound depend also on the weights of the respective members.

The base price at the mills of material of high-strength carbon steels is from 20 to 30 percent higher than plain carbon steel. The low alloy steels cost about 50 percent more than ordinary carbon steel, due to their higher cost of production.

The economy of using high-strength steel for a particular structure can only be determined by making cost estimates based on approximate designs for both kinds of steel. If very long spans are involved, or very high freight rates apply, the high-strength steel design is likely to be economical; also if there is a double saving when weight is reduced, as in the case of movable bridges and cantilever bridges.

Building construction generally is not suitable for the use of high-strength steels, although even here it may be economical when long roof or floor spans and heavy loads are involved. Tower struc-

tures are sometimes made with high-strength steel to save weight and cost.

If the saving promised by the use of high-strength steels for any type of structure is doubtful or small, most engineers prefer to specify A7 steel for reasons of rigidity and thickness of material.

REFERENCE

1. DE VRIES, K., "Strength of Beams as Determined by Lateral Buckling." *Trans. A.S.C.E.*, 112:1245 (1947).

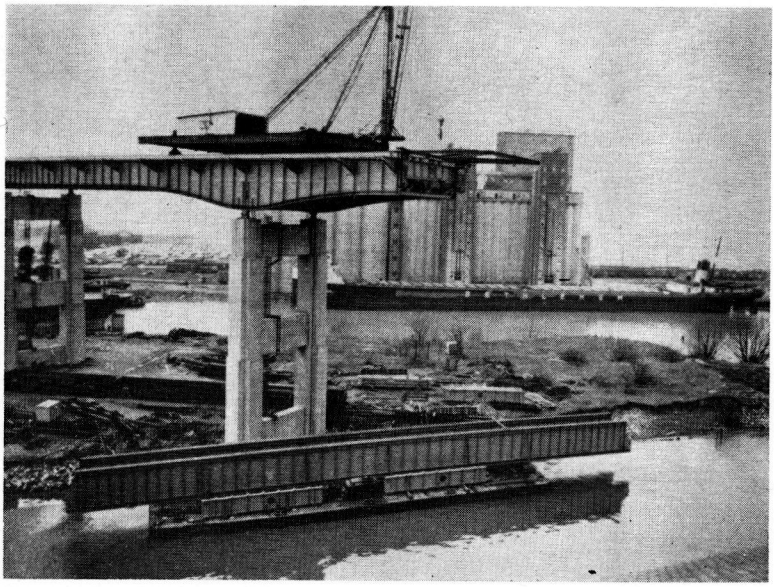

Figure 17. Typical cantilever erection of girder span to avoid falsework.