A Structural Future for Alloy Steels

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This paper reviews the reasons for the use of high-strength steel in highway structures. It points out the economies that can be realized in combination with welding to save steel weight, minimize secondary stresses, and improve aesthetic lines. Some of the uses that have already been made in California are briefly reviewed, as are factors that usually govern design and features that must be watched when combining several different types of steel. Shop and field procedures are covered, as well as some of the unusual effects that result from the use of these steels in large girders. Field welding procedure is reviewed. The benefits and limitations that accompany the use of high-strength steel, along with some predictions as to how far its use can go in the near future are discussed. The economics of the use of high-strength steel are reviewed, dealing with economical span lengths and sections and with the competitive place of steel with concrete in various spans. The paper concludes with a discussion of the problems of quality control and inherent consistency of physical characteristics which will be essential if these high-strength steels are to be generally accepted.

•NOT LONG AGO, when steel (especially steel for a highway bridge) was mentioned everyone knew it meant carbon steel, or mild steel (it was known by several names); today it is known as A7 steel by the ASTM designation. Of course, it was known that better steels could be made—high-strength steels for banjo strings, suspension bridge cables, or automobile springs. However, these were special steels, usually made in small, controlled quantities. When a bridge was built, plain structural steel was used. It could take a beating and was not too susceptible to damage in handling. If it was bent, it could be straightened. It could even be heated to straighten it with no harm. When welding was introduced, 90 percent of it was very weldable.

Times have changed. Now there is probably too much of everything without really knowing what one has. There are structural high-strength steels in all ranges up to a yield stress of 110,000 psi or so, alloy steels, heat-treated steels, weldable steels, steels that may be welded with care. Starting with a working stress of 18,000 psi for A7 steel, any working stress up to 50,000 psi can now be obtained.

The reason for these high-strength steels in structural work is that if a pound of steel can be made to carry 54,000 lb instead of 18,000 lb, one is three times better off. Of course, the price goes up too, but not as fast as the strength, so there is still an economical exchange. There are other incidental advantages. Besides the saving in weight of steel for the primary members, there is an over-all saving in deadload. There is a general lightening of the members which reduces the secondary stresses. Over-all, there is a general slimming effect that makes the structure look trimmer and better.

Beside the economical advantage of getting more strength per pound of steel, the use of high-strength steel offers other advantages to the designer. For one thing, it permits designs that would otherwise not be practical, such as reasonably long span parallel chord trusses and uniform depth girders. It simplifies the over-all design and makes these structures less complicated. Although the fabrication may be somewhat confused by several different types of steel, as a general rule, the over-all fabrication is simpler because the members are lighter and consist of fewer pieces. This,

of course, leads to a corresponding economy and greater ease in shipping and erection, as well as maintenance. It also follows that because these structures are simpler, cleaner of line, and less complicated, they are also more pleasing to look at and are worthy examples of the bridge builder's art.

There are many complications, however, that must be faced, and as more and more uses of high-strength steels are found, there must be a greater understanding of these complications and more uniform rules for dealing with them.

The first complication is that regardless of what different steels may be chosen for use, they all have the same modulus of elasticity. Deflection and buckling formulas are all functions of the modulus of elasticity. That means that the deflection and the susceptibility to buckling increase along with the working stress. This spoils much of the benefit to be derived from the higher strength.

A second difficulty in the use of the high-strength steels is the fact that the fatigue properties of the steels thus far developed do not improve with the breaking strength. For a zero-to-tension stress cycle applied to butt welds, the fatigue strength of a high-strength low-alloy steel is about the same as that of the same type of joint made of an A7 or A373 steel. In other words, if a variable-stress fatigue-producing condition exists, there is no advantage to using a high-strength steel.

A third complication is the difficulty of working with these high-strength steels. They are all heavily alloyed. Some are heat-treated. When these steels are welded, troubles arise, which though not unsolvable are nevertheless complications that make the fabricating and welding of high-strength steels a much more exact science than it has heretofore been considered.

High-strength steels are becoming more and more available, and there are definite advantages in using them. There are also some drawbacks.

From the myriad of steels available, it is first necessary to select a few in desirably stepped ranges to use in a design. So far, the California Division of Highways has limited itself to three steps and selected three steels that have been combined satisfactorily on several occasions.

To get a spread of strength that would make the changes in section and splices economical, A373 working at 18,000 psi, A441 at 27,000 psi, and T-1 at 45,000 psi were chosen. T-1 is now commonly given a higher working stress, but in the early days of investigation 45,000 psi was considered quite high enough.

This combination was first used on Carquinez Bridge. This is a large, double cantilever truss. Descriptions of the design of this structure have been thoroughly covered in the literature elsewhere so they are not repeated here. The use of these various steels at Carquinez was what one might now call almost conventional. When the stresses went high enough to make the A373 member cumbersome and heavy, the Division changed to A242. When the stresses rose still higher, the Division went to T-1 (see Table 1).

TABLE 1

ALLOWABLE STRESSES FOR THREE GRADES OF STEEL

Steel Grade	Yield Point	Tension	Compression
A7 and A373	32,000	18,000	$15,000 - 0.25(\frac{L}{r})^2$
A441:			
¾-in. and under	50,000	27,000	$22,000 - 0.56(\frac{L}{r})^2$
¾-in. to 1 ½-in.	46,000	24,000	$20,000 - 0.45(\frac{L}{r})^2$
1 ½-in. and over	42,000	22,000	$18,000 - 0.39(\frac{L}{r})^2$
T-1	90,000	45,000	$36,000 - 1.75(\frac{L}{r})^2$

Naturally, the purpose in using high-strength steels is to achieve minimum plate thickness. A uniform size member was selected, and then the minimum plate thickness was sought to make it carry the load. It did not take long to find that in many cases the minimum plate thickness rather than the stress was governing the plate selection. For instance, a T-1 plate would be adequate but, because its thickness was thinner than the acceptable limits, A242 would have to be used to gain the necessary thickness. Further, the same thickness requirements might force changing from A242 to A373. The thickness requirements are complicated by the variations in stress allowed for the different thicknesses.

One of the frailities of high-strength steels is the nearness of the yield point to the ultimate. To all intents and purposes, if a high-strength steel yields, it has failed. This is especially true in a truss. Knowing this, the question arises whether it is proper to use T-1 steel in gusset plates, for instance, where it is known there has to be some movement and some giving. In the panel point connection where the floorbeams tie in, is it safe to use these very high-strength steels, or should the high-strength steels be used in the pure stress applications and a lower-strength steel with more plastic capabilities be used where it is known bending and giving must take place? Time will have to tell. Though T-1 gusset plates have been used, there is no evidence of any difficulties at these points but pure theory raises the question.

A second use of high-strength steels in combination in one structure was in the Benicia-Martinez Bridge a few miles up the Straits from the Carquinez structure. This is a bridge well over a mile long, of which nearly 4,900 ft are ten spans of continuous deck trusses. The mixture of the different steels in these trusses was also somewhat elementary (see Table 2). The T-1 steel was used extensively in highly stressed members and in gusset plates. A 242 steel was used to meet intermediate stresses.

TABLE 2

TONNAGES OF DIFFERENT STEELS USED IN THREE BRIDGES

Bridge	Steel (tons)		
	A373	A242(A441)	T-1
Carquinez [a]	6,440	5,370	2,910
Benicia-Martinez [a]	5,810	1,800	3,580
Whiskey Creek	200	238	390

[a] Trusses only.

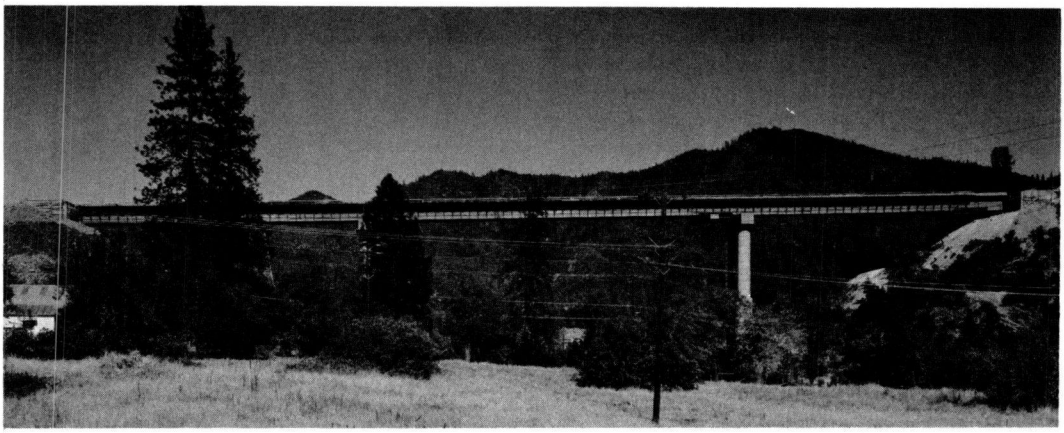

Figure 1. Whiskey Creek Bridge.

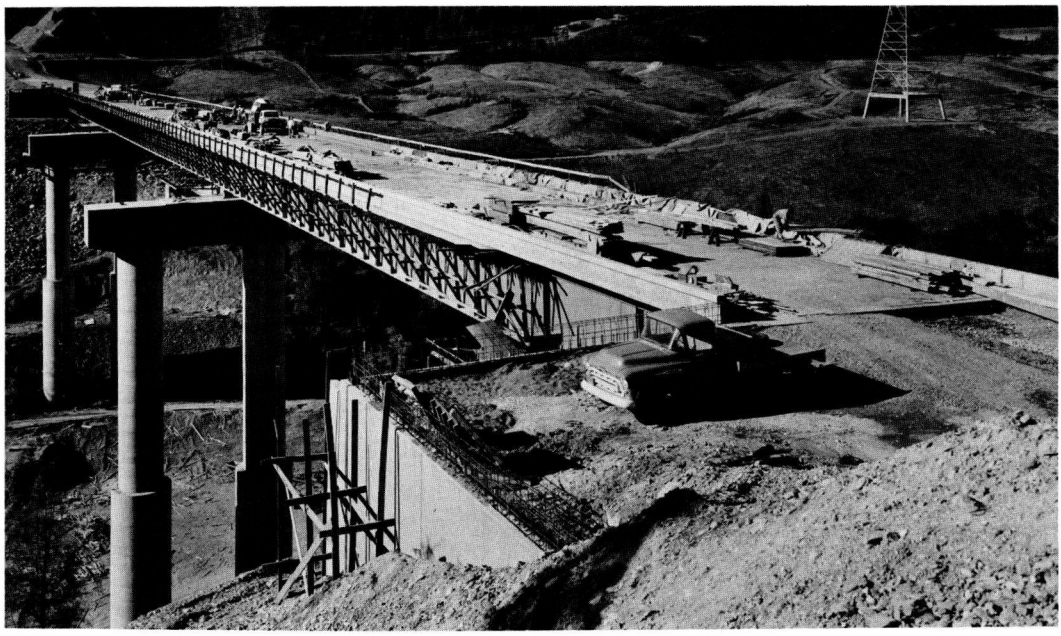

Figure 2. Whiskey Creek Bridge. Piers provide for future widening after reservoir is filled.

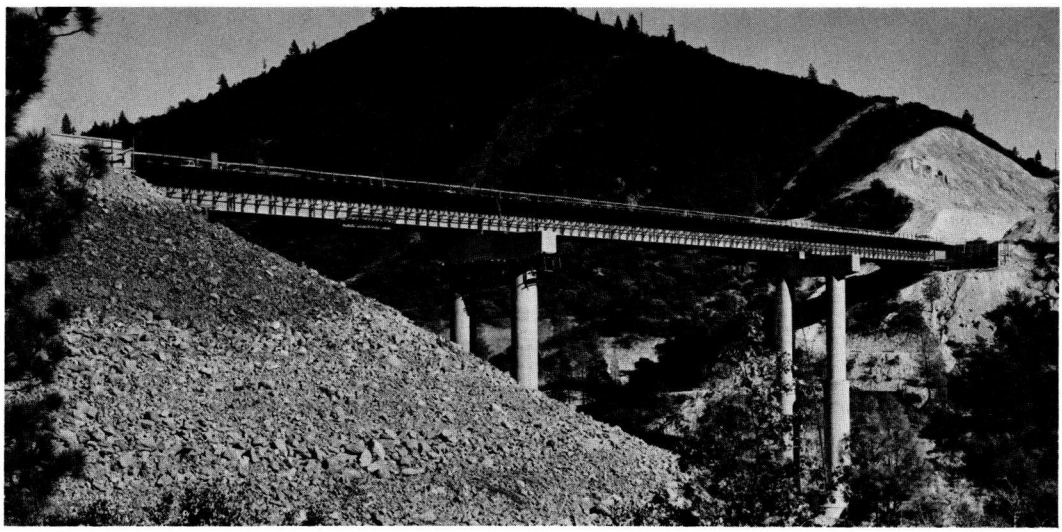

Figure 3. Whiskey Creek Bridge. A welded plate girder design using three grades of steel: A373, A441, and T-1.

Another extensive mixing of these high-strength steels, which also has had ample publicity, is the plate girder bridge at Whiskey Creek (Figs. 1, 2, 3). This was a different situation than the big truss at Carquinez. Here, three different grades of steel were used to fabricate a deep plate girder, splicing in the steel types as required to meet the stress requirements.

The bridge is about 870 ft long. It has a center span of 350 ft and two end spans of 260 ft each. The web plates are uniformly 12 ft deep and ¾ in. thick throughout. Both flanges are uniformly 1 ¾ in. thick except for a short section in midspan which has to be 2 in. thick.

At Whiskey Creek, the structure had many incidental advantages. First, a girder of uniform depth was developed throughout. The web plates were all of the same depth, making the fabrication easy. There were no stress-raising steps in the web where the flange changed thickness. This also led to a very different appearance. Years of structural design have conditioned one to expect heavy cover plates and thick flanges near the centers of the spans. Here a girder of constant depth with trim-looking flanges extending from one end to the other was used.

Another advantage was that the contractor developed travelers that ran on the bottom flanges. These were used for setting and removing falsework as well as for painting. The smooth flange made an ideal rail.

The trim lines and smooth appearance did not happen accidentally. Considerable work and planning went into making the structure come out as it did. As often happens, some of the simplest appearing things are the most complicated.

The most difficult part of the design of any of these mixed-steel structures has been setting the ground rules. There are many people and agencies that must be satisfied, and in many phases of the design work with high-strength steel, there have been no design rules or guide posts previously established. This is still the case. The specifications of the American Association of State Highway Officials are of little assistance with the higher-strength steel. The American Welding Society specifications are very limited in their scope as they apply to these steels. Many of these steels do not yet have even the definition of an ASTM designation.

On this uncharted sea, the designer must lay a course. Usually he is limited as to time so he cannot treat it as an academic project for unlimited research but must turn out a usable set of plans in a reasonable time. He must extrapolate formulas far beyond their accustomed limits. He makes far-reaching assumptions and uses liberal amounts of horse sense. Ultraconservatism is often the safeguard against ignorance.

On the Whiskey Creek Bridge, the sequence of design was as follows. First, the approximate design criteria was determined for the use of T-1 steel in the plate girders. Next, assurance was obtained from the Materials Laboratory that these three types of steel could be readily welded together to develop a strength at the weld equal to the strength of the lesser of the two steels being joined. This, they felt, was quite possible with carefully controlled welding processes; the actual fabrication of the structure has proved this to be true.

Next, a preliminary design was run out, and the structure was estimated. At this stage, several different arrangements of the three steels were studied (see Fig. 4). Comparisons were made with other possible designs that would be satisfactory in this location. More work was then done on the ground rules to determine design criteria for the final design, including the stresses at which the steel could be worked, allowable stresses for the flanges, both tension and compression, the minimum web thickness that could be tolerated, and the proper formulas to be used for investigating the buckling propensities of the proposed beams.

The design calculations were finally made. These calculations were actually no different than those required if design had been done entirely in carbon steel. It soon became evident from the calculations that the liveload deflection was going to control the design of these light and flexible girders. From deflection limitations, a minimum girder section was established.

In the end, both the flange plates and the web plates were made up of all three different types of steel. The flange plates were of uniform width and thickness throughout except for a short section in midspan where the thickness was increased. The web

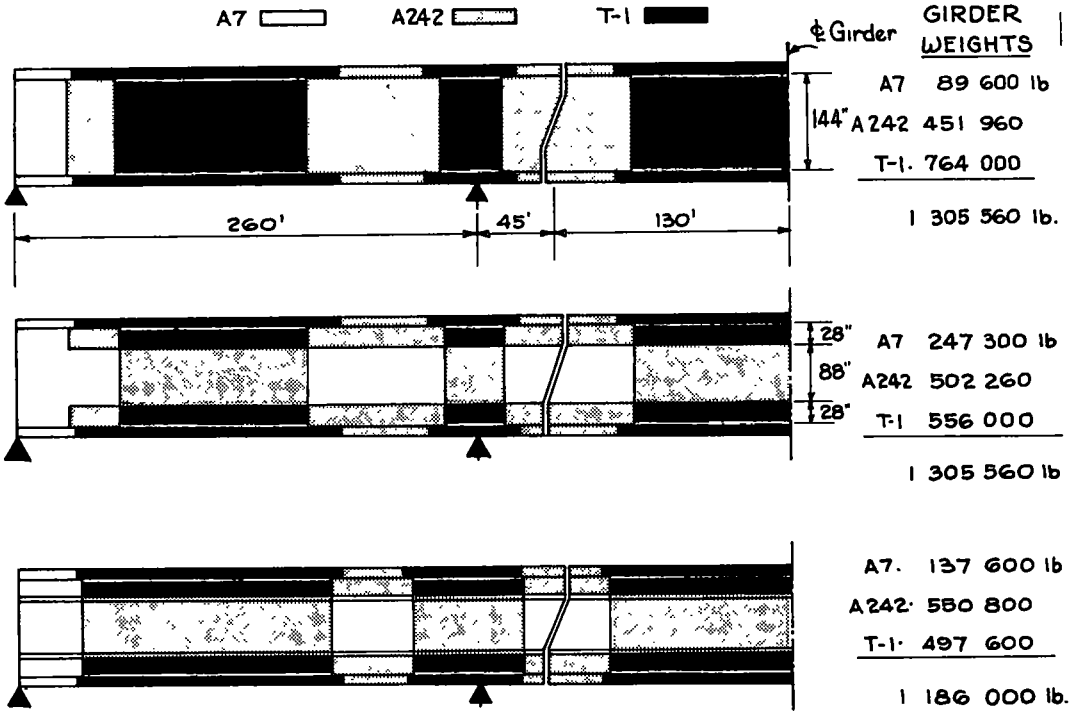

Figure 4. Alternate girder designs for Whiskey Creek. Top design was the one used; the other two alternates, though economical, required more expensive fabrication.

plate was also made a uniform thickness and depth for the full length of the bridge. Some steel could have been saved by making the web plate thinner in some sections. This would have been possible only because of peculiarities in the specifications. It was felt that it was better to maintain the slightly heavier web throughout rather than risk a girder that might become too flexible.

This matter of deflection had another interesting result. Temperature deflection formulas, like load deflection formulas, are dependent on the modulus of elasticity of the steel. In this respect, a high-strength steel bridge will suffer just as badly from temperature differential as will an A7 steel bridge. In fact, the effect may be somewhat greater in magnitude. Being light and thin of section, these girders change temperature rather rapidly. At Whiskey Creek, as soon as the sun hits the girders, they begin to sag. This brought considerable distress to the field engineers who take pride in the perfect alignment of the bridge railing. Although the railing will be perfectly straight early in the morning, by noon, when the sun has been on the girder sides for several hours, there will be a deflection of several inches in the side spans and the center suspended span and a rise of one to two inches in the hinge points in the center span. Some sideways bowing of the girders is also apparent. As yet, the structure has not experienced the hot summer sun, so the full extent of these deflections is not yet known.

Had it been realized how susceptible the structure was going to be to temperature changes, the rail could have been set for some intermediate temperature. This is something to think of in future designs. Actually, a deflection of even 3 in. in a 350-ft span is apparent only to those engineers and observers of a critical bent who habitually squint along every bridge rail to see how good a line was obtained during construction. No adverse structural effects are anticipated from this temperature movement. This is a side effect that may be anticipated from lightweight high-strength designs.

To digress here for a moment to point out some of the idiosyncracies of current specifications and to illustrate some difficulties they can cause, having different stresses allowable for different thicknesses of steel can become bothersome and result in some strange steel combinations. One wonders if they are completely necessary. For instance, in A441 (or A242) steel, for thin plates ¾ in. and under, the yield point must be 50,000 psi, for which there would be a working stress of 27,000 psi. Between ¾- and 1½-in. plate, the yield stress must be 46,000 psi for which there would be a working stress of 24,000 psi. At the other end of the scale for heavy plates over 1½ in. up to 4 in., the yield strength must be only 42,000 psi for which there would be a working stress of only 22,000 psi.

As is the case with any stepped specification, the results are strange and somewhat irrational adjacent to the step. However, in this case, where steels of different types are being mixed, a new problem emerges. If it is assumed that a flange 1 ¾ in. thick is being joined to a web ¾ in. thick, the working stress for the entire section is slightly less than 27,000 psi. An A242 steel can be used for the web plate because it is only ¾ in. thick and the allowed working stress for that thickness is 27,000 psi. However, for the 1 ¾-in. flange plate, the allowable working stress for A242 is only 22,000 psi, so it is not strong enough. Therefore, to maintain the flange at the uniform 1 ¾ in., for the flange plates, a higher-strength steel (in the Division's case T-1) has to be used to meet the 27,000-psi stress.

This is the explanation for some misunderstanding which has developed to the effect that high-strength steel flange was used along with a lower strength steel web. It is purely mathematical; the steel must be chosen to meet the stress. Because of the step in the specifications, the results look odd. Here again, this illustrates the involvement that comes from designing from steels of three such widely separated stress grades as 18,000, 27,000, and 50,000 psi.

These variations in stress with different thicknesses are not just theoretical limits. These are actual physical differences which occur in the different thickness of steel due to the working the billet gets during the rolling process. Because the variation is a function of the thickness, the idea has been proposed that instead of the stepped specification, there should be a curve in which the allowed stress would vary directly with plate thickness. The steel producers do not view this idea with much favor, however. This is another case where some modification of long-established mill practice could work to increase the availability and use of steel.

When increasing stresses force a change from the 27,000-psi grade to go to a higher strength steel, the 50,000-psi grade, if it is used, is underworked by a rather wide margin. This points up the need for another fourth step which would have a working stress of around 36,000 to 40,000 psi. Given four steps of about equal range from 18 to 50,000 psi, the designer can make much smoother transitions and much more economical use of his steel. However, from a manufacturing standpoint, metallurgists would have to put so many alloying elements into the steel to get the 36,000-psi grade that they might just as well add a little more and get the 50,000-psi grade. This may all be perfectly true, but it is something for the steel industry to work on.

In order that these high-strength steels may successfully and economically be used, there should be some industry-wide effort toward grouping them into standard steps. It would be chaos for a designer to attempt to design and call for a different steel for each stress he encountered.

A designer can conceivably design each different part of a structure for a different strength steel. It would be possible to obtain a type of balanced design in this manner and still maintain rather uniform-looking sections throughout. The plans could clearly show what steel is to be used for each member: 18,000-psi steel for this one, 27,000 psi for that one, gusset plates of 36,000 psi, diagonals of 50,000 psi, and so on. However, there would be great confusion in the shop. Pieces of steel whether they be a common A7 variety which yields at 33,000 psi or a specially treated type which will go to 150,000 psi, all look the same. To the man in the shop, there are endless opportunities for getting confused. Mistakes do occur. There have been some embarrassing experiences with prestressing steel where some mild steel bars got mixed in with the high-strength bars.

There is great need for positive and uniform methods of identification of these various steels. This has been suggested to the American Institute of Steel Construction, an association of steel fabricators, as a worthwhile project for their group.

Though a mixed design sounds chaotic, it is caused by the many different steels which are available. Each steel company has a few, covering a wide range of strengths—with several more in development. These are not grouped or stepped but cover the full range only a few thousand pounds apart. The public agencies especially must write their specifications so that several different steel suppliers may meet them competitively. A specification cannot be written to use exclusively some special steel that a company produces. The solution is for the various steels to be grouped by reasonable steps that the designer can use, knowing there will be competitive steels available on each step.

There are too many steels available. It seems proper to look to the steel industry to get together and offer the designer a wide variety of steels all grouped into, say, four definite stress groups, expanding, if necessary, to five or six as the steel strengths go on higher and higher—then the designer can have an orderly progression from which to choose and can make a reasonable and economical design. In the face of this multiplicity of choice, the Division has stayed with three basic stress groups.

In the case of Whiskey Creek, the use of three different steels made possible a plate girder of uniform depth. This had many advantages. It simplified the fabrication and enabled the contractor to build a traveler that ran easily on the bottom flanges. It also made possible a rather simple method of erection. Without heavy cranes or equipment, these two girders were rolled out from either end and then, on jacks, dropped into place. Had the girders been of varying cross-sections, even of varying flange depths, or had they been deepened at the haunches over the piers, as is commonly done, this simple method of erection would not have been possible.

The uniform depth has other benefits in that it is also easier to handle and ship, and in the last analysis, creates a better looking structure.

Earlier, it was mentioned that the fact that all steels have the same modulus of elasticity spoils many of the benefits one would expect to get from high-strength steel. This is well illustrated in the Whiskey Creek design. Regardless of the high stresses at which the steel could be worked, it was found that deflection was dictating the size of the cross-section. The AASHO specifications, which are very reasonable, limit the main span deflection to L/800 and the cantilever deflection to L/300 (L = span length). This limited the deflection in the 350-ft span to 5.25 in. and the cantilever arm to 1.8 in. To meet these restrictions, it was necessary to reduce the fiber stress in the flange plates from the allowable 45,000 to 39,700 psi in the area of negative moment near the pier. This brought the live-load deflection within the allowable but it is easy to see that the capabilities of the high-strength steel have been seriously curtailed.

The arbitrary limitations on depth-to-thickness ratios of plates to prevent buckling also make it impossible over much of the girder to work the plates at their maximum stress. The AASHO specifications set up a maximum depth to thickness ratio of 340 for A373 steel and 280 for A441 steel. There are no established limits for the higher-strength steels. By extrapolation a value of 200 is obtained for T-1 steel (Fig. 5). To make sure the AASHO formulas were not being pushed too far, the Division checked the plates by buckling formulas and found the assumptions to be safe (Fig. 6). This is an uncharted zone, however, but it seems quite certain that the present application of these limits is well on the conservative side.

Because the web buckling proved to be a controlling factor in limiting the web thickness, it is desirable to have more information and useable relationships for this characteristic in high-strength steel. If the relationship shown in Figure 5 is stated another way, formulas may be developed that define the specification limits and may be logically extended into the higher ranges. Such a relationship is shown in Figure 6. By plotting the AASHO specification limits and passing curves thru them, two rather simple relationships are developed between the ratio of girder depth to web thickness (D/t), and the actual compression stress in bending (S).

For webs with transverse stiffeners only,

$$\frac{D}{t} = \frac{22,500}{\sqrt{S}}$$

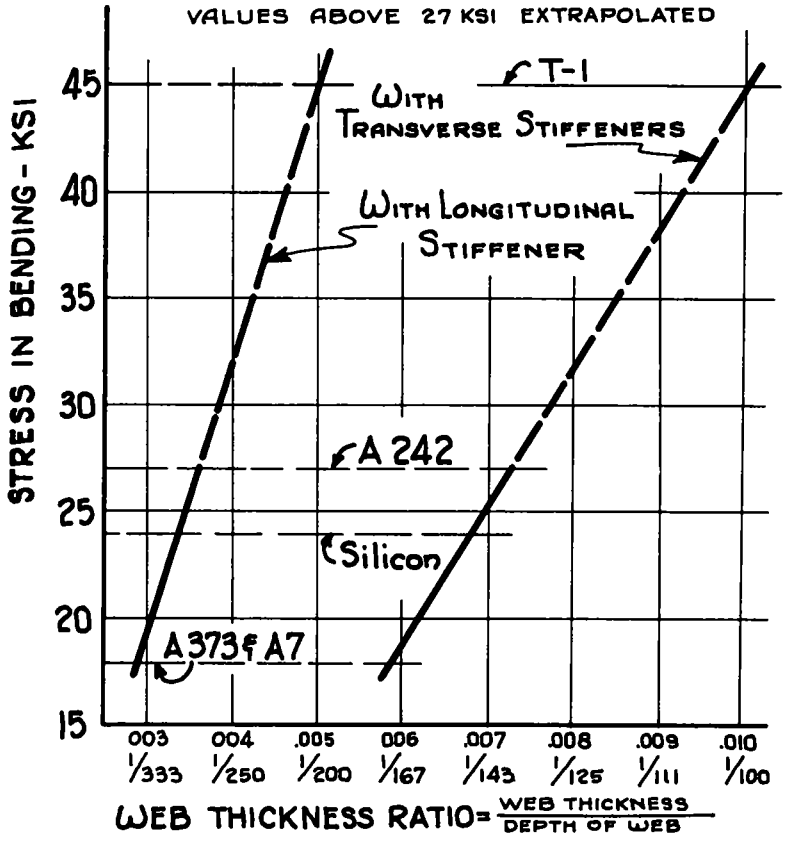

Figure 5. AASHO web plate thickness limitations.

For webs with both transverse and longitudinal stiffeners,

$$\frac{D}{t} = \frac{45,000}{\sqrt{S}}$$

The Whiskey Creek webs were well on the conservative side of the curve. More exact information on buckling of high-strength steel should make possible somewhat higher stress allowances and lighter structures.

The second of the difficulties that arise with high-strength steel came from its poor fatigue characteristics. Because only rarely are highway bridge structural members subjected to fatigue producing stresses, it is easy to become somewhat complacent in the consideration of the fatigue properties of the steels used in highway bridges. However, because fatigue seems to be more critical with the high-strength steels, it must be intelligently dealt with if these steels are to be used in structures.

A great deal is yet to be learned about fatigue effects. This very ignorance, coupled with some vague specification references to greatly reduced stresses where fatigue may be a factor, has caused some designers to shy away from the material. An understanding of the specific nature of the fatigue weakness as well as an analysis of the likelihood of ever experiencing the critical number of repetitions should allay some of these fears.

Munse and Stallmeyer in their tests at the University of Illinois found that for steels in the 60,000- to 80,000-psi ultimate strength range, a zero-to-tension stress variation would require a working stress reduction to about one-half (0.512) for a 100,000-cycle frequency and to about one-third (0.345) for a 2,000,000-cycle frequency. Their tests emphasize the fact that the high-strength steels are very sensitive to physical defects,

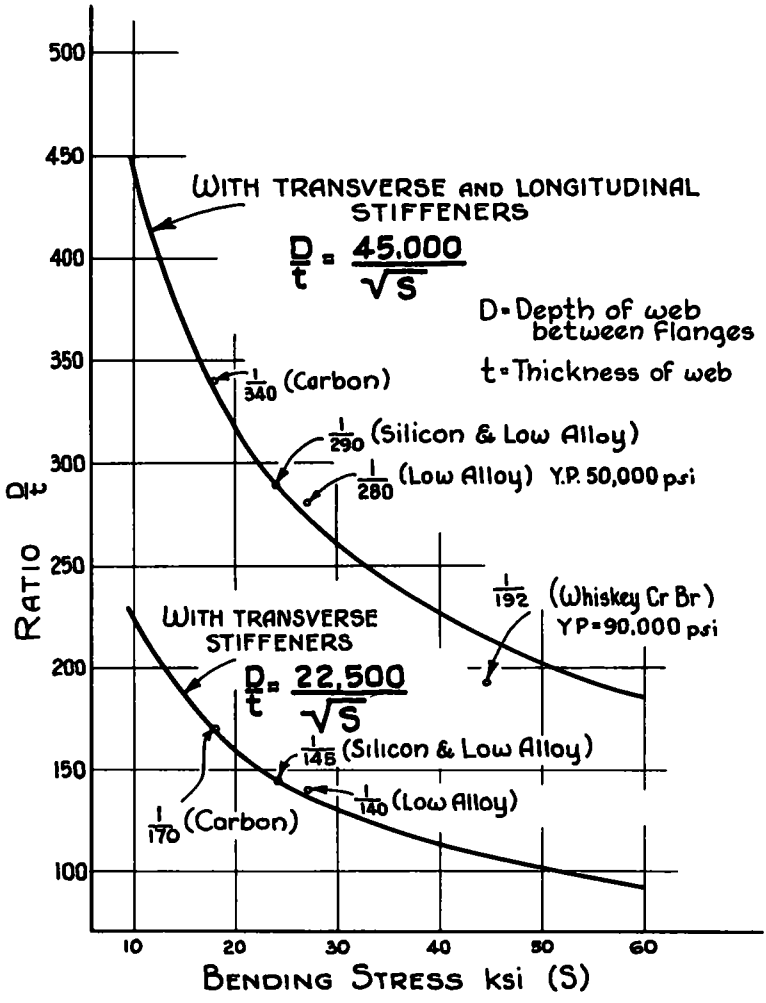

Figure 6. Web thickness of plate girders (developed from AASHO specification).

notch effects as a result of shape, and all manner of welding defects. Their tests and conclusions merely reiterate that this is a serious obstruction in the path of high-strength steel use. It must be met with more research and more information so that it is known rather than assumed.

Few highway structures contain members subjected to zero-to-tension stresses. Most probably come within a range of 50 percent stress to full tension. Information is inadequate for this range.

On the theory that structures are being built to last forever, the 2,000,000 cycles are often chosen as the conservative figure of what the structure must undergo. If this figure is used, it means that during its life, the structure will have to undergo, on 2,000,000 different occasions, a stress reversal of from zero to full stress. This means that the structure must be subjected to this zero-to-maximum capacity load once each hour of every day for 228 years. If the figure 600,000 repetitions is used, it would be assuming an hourly maximum loading for over 68 years. Even the lowest frequency, that of only 100,000 cycles, would allow a maximum loading about every 4 ½ hr day and night for 100 years.

These figures have little practical value. They only serve to reduce to understandable terms what is meant by cycles of repetition. Knowing that the average highway

structure may receive its full design load only a few times during its entire life, even the 50 percent stress reduction for the 100,000 cycles may be ultraconservative. On the other hand, some of these steels may become sensitive to a variation of 50 percent-to-tension or 75 percent-to-tension. These facts are needed before this reduction can be pared down to its lowest safe value. Until the time when much more information is available about the fatigue effects at various combinations of stress variation, these high-strength steels are going to be perhaps unjustly handicapped by the allowances that must be made to compensate for what is not known.

Next, some of the difficulties of handling and welding these different materials in the field are examined. Because the clean, trim lines that would result from the uniform thickness of flange and web were sought the field connections were not bolted, but rather all were welded.

Field welding of splices is not always the most economical method. This is especially true when the job is back in the mountains, away from population centers, and in a warm climate. Whiskey Creek had all these characteristics. There was another job recently, in somewhat a similar location, in which the contractor had the option and chose to weld the splices and save the cost of the splice plates and bolts, but he afterwards regretted his choice and said the bolted splice would have been much cheaper. Opinions on bolted or welded field splices vary widely with the location and the fabricator so that no general conclusions should be drawn. At Whiskey Creek, it was required that the splices be field welded.

The girders were delivered to the job in 76- to 108-ft pieces. Then they were set up and carefully aligned to the position from which they would be rolled or lifted into place. The two girders were also tied together by cross-frames and lateral bracing. At each joint, there were two flanges and a web plate to be joined. These flanges were 30 in. wide and 1 ¾ in. thick, except for a 90-ft section in the center span which was 2 in. thick. The web plate was ¾ in. thick and 12 ft deep. An offset of 1 ft was made between the splice in the flange and the splice in the web. All of the edges came to the job beveled and ready for welding. The flanges were butted and welded first. As the flanges cooled and shrinkage resulted, the web plates came together and in many cases actually slipped by each other. As this happened, a cutting torch was run down the slot to establish a ⅛-in. open crack. When the flange welding was finally completed, a final pass was made down this slot with a torch to obtain the required clearance. Then the edge of the cut plate was scarfed in preparation for the welding. The amount of this shrinkage varied considerably so that it was not felt feasible to fabricate the girders in the first place with the web plates cut back the required amount.

As to the precise welding procedure, both ends of the flange joint were preheated 350° to 400°, 1 ft back from each side of the joint. The bottom flange was welded first and then the top flange. About 1 ft of the fillet weld connecting the web to the flange was temporarily omitted at each end to allow for the movement when the shrinkage caused the web plates to slip by each other. Just before the web was welded, the flanges were preheated and the ⅛-in. clearance was again established between the ends of the web. The webs were then preheated, causing them to come together again. The web was then welded and preheat was maintained in both flanges and webs. As the girder cooled down, the stresses were properly distributed. If the web is not kept free until the flanges are welded, in a manner somewhat as described, or should the web be welded first, the web will buckle with a typical oil-can type of buckle.

While the flanges were being welded, there were four torches playing on the flanges. To maintain the preheat, there were two torches on each side of the joint about 1 ft back. With this procedure, successful connections were made between T-1 and A242 steel.

In spite of the advances in metallurgy, even utilizing all of the recent advances in technique, and in rods and wire and fluxes, in the welding of high-strength steels, the welder, the man controlling the process on the job, is still of great concern to the manufacturer or fabricator. The welder must exert a great deal of personal skill to make a weld good whether it be in the shop or in the field.

The welding of high-strength steels requires more training for welders. It requires a great deal of skill, not a mere short course in general welding. The Division has welded a great many bridges and the pattern always seems to be the same. The start

of a job resembles a welding school more than it does a construction project. During the welder qualification period, supposedly competent welders are found doing every conceivable thing wrong—wrong polarity, wrong bead forms, wrong procedure. Even after being qualified, many of them seem to resent the strict and careful procedures that have been found necessary. Constant and close supervision is necessary. All of this makes the work very difficult and expensive for a contractor who is facing a time deadline and cannot get good welders.

In the shop, it is possible to use jigs and clamps and automatic machines that do a marvelously uniform job. Even here, there are chances for error and failure. The welds must be clean. The old fabricating shop with its leaky roof and dirty floor must be cleaned up if they are planning to weld some of the more exotic steels. Those doing the work must have a thorough understanding of what they are doing.

There is the matter of tack welding, for example. As a holdover from welding A7 steels, it was, until recently, common practice to clamp up the plates to make a member and then tack weld them together to hold them in position. However, with the high-strength steels, tack welds are just as important as major welds. Unless they are given the same preheating care planned for the major weld, these tack welds can be a source of trouble. But these things are not easy to impress on someone who has been welding for 10 or 15 years.

Out in the field, it is even more difficult. In the first place, it is hard to get welders onto the job. Whether it is a bridge in the mountains or a missle base in the middle of a prairie, it is hard to attract good men. Various promises of overtime, long hours, transportation, and fringe benefits are necessary to get the men to come out to the job. Then, when it is considered that of thirty so-called welders tested, one will be lucky to get two who can qualify for welding high-strength steels, the problem assumes even greater difficulty.

Welding is hot work, the weather is usually hot in the summertime, many welders seem almost happy when they are thrown off a job as being incompetent. The main hope and source of strength seems to be in a factor of psychology. The welding of these high-strength steels has become such a challenge that welders who take their work seriously like to become known as men who can handle these fancier jobs. As a result, some were found who will stick it out and try to do a good job to get the experience.

The fact remains that good welders are scarce. Through this last summer and fall, there was advertising over the radio for welders to work on the atomic submarines being built at the Mare Island Navy Yard. The Navy Yard too is working with high-strength steels, and no doubt their ratio of good welders to the number that apply is probably about the same as the Division's

This scarcity of good welders has an effect on the price, quality, and general desirability of welding. It is one of the best reasons for doing all the welding possible in the shop and holding the welded field splices to a minimum. This dependence on human fallibility is one of the problems that has to be borne. Defective rivets can be cut out. Loose bolts can be tightened. Bad welds take more complicated and expensive repair. Although this is a discouraging factor, it should not cause anyone to decide against the use of welding. With proper planning of the procedure and heating requirements, with intelligent inspection during the operation, supplemented by radiographic and other forms of examination, good welds can be obtained.

The mention of inspection of welds opens up an area too broad to be covered under this subject. However, the future of high-strength steels is inseparably interwoven with welding. The dependability of welding in turn can only be assured by adequate inspection.

High-strength plates, in themselves, give some trouble in fabrication. In every case so far, there has been some difficulty with the flatness of the T-1 plates. A fabricating shop, in accepting a T-1 job, must realize that here is a tough steel which must be straightened cold. No longer can an artist with a torch be turned loose to iron out the kinks. In addition to general warpage often present, distortions may be accentuated by flame-cutting a large plate. There seems to be some relief of stresses locked in by the heat-treating operation.

On Whiskey Creek, there was also trouble with the welding causing warping in the flanges When they got to the job, it was found that the bearing areas of the bottom flanges were warped as much as $^3/_{16}$ in. This necessitated making special bearing plates to fit the bottoms of the girders.

The higher the strength of these plates, the more complicated the formulation, the more extensive the heat treatment—these all serve to make the steel itself more precise in its manufacture. They also serve to make it demand more precise handling in the fabrication process. This is something apparently not generally recognized. Certainly most welders are not aware of it and many fabricators do not realize the precise nature of the operation. It has many of the aspects of a laboratory operation. However, if established procedures are followed, if good workmanship with proper electrodes and fluxes are used, if special attention is given to moisture control of electrodes and fluxes, and if the material to be welded is properly prepared so it is free from moisture, rust, dirt, grease, and mill scale, then a reliable weld can be made.

It was recently said that one gains from his own or another's experience, but only if one will observe and profit by that experience. The facts of how high-strength steel must be fabricated are becoming evident. Yet a surprising number of fabricators must be shown, must learn the hard way that these restrictions also apply to them.

These restrictive practices, which must be followed to weld these steels, naturally have a detrimental effect on the economics of the use of the high-strength steel. In designing a structure and planning to combine several of these steels, one must take a hard look at the costs of all these splices.

Some preliminary studies (Fig. 1) were made in which a girder was cut up like a jigsaw puzzle and different grades of steel put in the proper places to match the stresses. From a theoretical standpoint, the design was excellent. Every pound of steel was working at a high percentage of its allowable. But when the number of splices; the numerous and extensive welds that would be required, the fitting, holding, and shrinkage problems were examined, it was easy to see a beam which was too badly cut up was just not practical.

This is the problem every designer must face. He will try to use the steel to the best advantage but he must balance his savings in weight and thickness of plates against a cost of making all these splices. There are no good general rules. It will vary with the area of the country, the availability of the steel, and the know-how and willingness of the fabricators and welders available.

Only a few years ago high-strength steels were undreamed of. Now, through the stimulation by space exploration and atomic research, they seem on the threshold of even greater advances. Metallurgical progress, increased research, and expanding space horizons make more and better steels with which to work inevitable. It would be unimaginative to think that the headlong progress of the past few years will not continue. With recognition of the inevitability of greater use of high-strength steels, it is necessary to keep pace with the materials that are being used. Perhaps even more important are the revisions of thinking and standards of quality so that this high-strength steel will meet the high degree of perfection required.

The problems arising from the fact that all steels have the same modulus of elasticity have been discussed. With the better steels, the strength increases but the resistance to deflection and buckling do not increase in the same measure. The higher strength the steel, the closer together become the yield point and the ultimate breaking point. This casts doubt on the wisdom of working stresses based only on the yield point. Controls have had to be based on the breaking strength, and thus far a safety factor of about 1.85 has been prevalently used. There has been some feeling that the higher strength steels should have higher factors of safety. Theoretically this point of view does not appear sound, but practically, there is some realistic thinking behind the idea.

The highest strength steel the Division is currently using in structures is the wire for prestressing. Ultimate stresses well over 200,000 psi are common. The smallest defects in this steel have been found to cause failure—a nick, a corrosion pit, or a splatter from a welding arc. The higher the strength of the steel, the more importance its defects assume. This is equally true of plates and structural sections.

If these high strength steels are to be turned out by the same processes and with the same degree of inspection and care as an A7 structural steel, undoubtedly there will be trouble. Small defects assume major importance. A bit of slag rolled into a billet, a billet end improperly cropped, a defect improperly repaired—any of these can cause a stress-raising defect that will start a failure in a high-strength member. One does not need much imagination to foresee what the failure of a critical member could do to the progress of high-strength steel development.

Each new application of these higher-strength steels is being critically watched. If a little trouble develops, the word gets around very rapidly. A bad failure could set the enterprise back years in its development. The course is therefore clear and, for those who are moving into new and untrodden areas, caution and conservatism become essential.

A similar concern and interest in quality is expected from those who manufacture the steel. As stresses go higher, defects become more important and less tolerable. This will necessitate new steps to improve on the milling process and remove the possibilities of human error which can cause defects in the finished steel. This is coming, but slowly. The very slowness contributes to the engineer's caution in using the steel. As stresses go higher, the quality controls must likewise be tightened.

The progress of the welding art is keeping pace with the steels in demand. Proper technique still involves considerable care and skill, but this too will be improved.

The present trend is toward automation. Cars are becoming so automatic that the art of shifting gears was almost lost until sports cars came along. A modern camera is valued because all one has to do is aim and shoot; no brains required. People expect welding to be the same way—automatic. Maybe someday it will be that way, but that time is not yet. A great deal of care and attention is required and if there are to be welded high-strength steel structures, that care must be given. With care, it can be done. Without due care, trouble inevitably results.

Looking back at the years of slow progress in steel development and then the hurried advance of the past few years, it seems certain that this is only the beginning. To meet this challenge, three things seem evident: First, more needs to be known about the characteristics of these steels. Second, this knowledge must be translated into new design criteria to guide progress. Third, as the stresses get higher, greater quality controls of the material itself are needed. Having these and guided by a progressive philosophy, advantage can be taken of these coming opportunities to make even better and more efficient use of steel.