

HIGH STRENGTH REINFORCING STEEL FOR BRIDGES

R. S. FOUNTAIN

Bridge Engineer, Portland Cement Association

Introduction

During the past ten years, greater advances have been made in the design and construction of concrete bridges than in any other comparable period. Development of prestressed concrete, improved precasting techniques, and dependable lightweight aggregates have played prominent roles. Notable contributions have also been made in reinforced concrete. Allowable stresses in concrete and reinforcing steels have increased, and distribution factors have changed. Fully integral concrete bridges with lengths of over 450 ft. are no longer unusual. Strengths of field-placed concretes average over 4,000 psi, and strengths of over 6,000 psi are commonplace.

Significant advances, of perhaps greater importance, will almost certainly take place in the next ten years. Ultimate strength design will probably be one of the early steps, with the recognition of moment redistribution expected to follow. New casting techniques and new shapes in prestressed concrete will improve economy in precast construction. For cast-in-place concrete the use of high-strength reinforcing steels, combined with the higher strength concretes readily obtainable, promises to provide an important change.

Economy

High-strength reinforcing steels cost appreciably less per ton of load-carrying capacity than other reinforcing steels now in use in bridge construction. Figure 1 illustrates the relative cost per ton of load-carrying capacity of three ASTM standard high-strength steels compared with the presently used intermediate grade steel. ASTM A432 steel with a minimum yield point of 60,000 psi has the

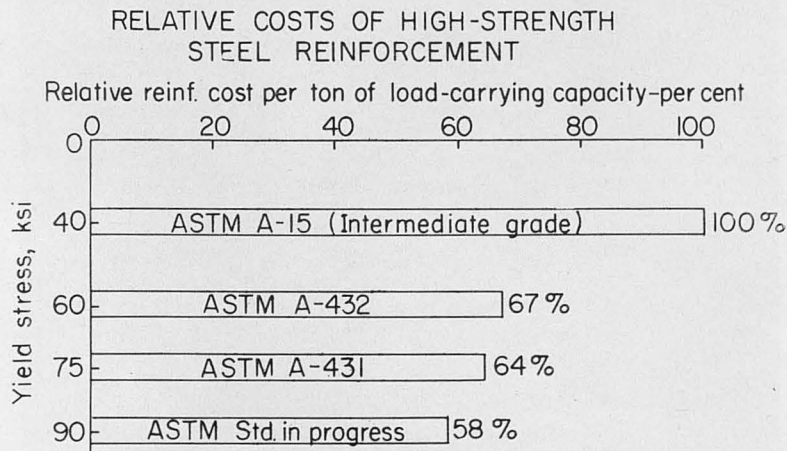


Fig. 1

same cost per pound as ASTM A15 steel, yet will carry 50 per cent more load. Therefore, the cost per ton of load-carrying capacity for A432 will be 2/3 or 67 per cent the cost of A15. Cost premiums will be required for the A431 and other higher-yield steels. Even with the added premium, the cost of the steel per unit of load capacity will decrease with each increase in yield.

The reduced steel cost is only a part of the overall savings that can be gained by using high-strength steels. Equal or greater savings can be obtained in the quantities of concrete required for beams. As an illustration, the excessive widths of T-beam webs required to provide room for placing the reinforcing steel will no longer be necessary. Less steel means thinner webs, reduced concrete quantities, and reduced dead loads, with more economical structures the end result.

Basic Design Requirements

Bridges using high-strength reinforcing steels and allowable design stresses varying from 30,000 psi to 50,000 psi have demonstrated the possible economies in Europe. However, the allowable stresses have not been the only criteria considered in the designs. Although we have incomplete knowledge of the calculations, there is little doubt that other requirements governed in many cases.

In all reinforced concrete structures, four basic requirements must be fulfilled to insure satisfactory performance. These requirements are:

1. Adequate safety against static failure.
2. Adequate safety against fatigue failure.
3. Satisfactory crack formations.
4. Satisfactory deflections.

In the past, these four basic requirements have been adequately met in this country by holding design stresses in the reinforcing steel to a relatively low value. The design stress of 20,000 psi allowed by the AASHTO Specifications for intermediate and hard grade steels has been found to give safe and dependable results in a variety of structures. It has been unnecessary to give specific consideration to ultimate static or fatigue capacity; satisfactory crack formations were almost always obtained with adequate design and good detailing; and deflections could be reliably estimated by computations based on the gross concrete section. All of the desirable qualities were virtually assured with little or no consideration given to volume changes in the concrete. In effect, the 20,000 psi maximum stress limitation has been a design panacea for reinforced concrete.

The minimum yield stress of intermediate grade reinforcing steel by ASTM specifications is 40,000 psi. With this yield, the design stress of 20,000 psi results in an overall factor of safety against flexural failure of about 2.1, as will be shown later. This is reasonably economical for medium and short spans. However, if high-strength reinforcing steels are to be used to economical advantage, allowable stresses must exceed the present 20,000 psi limitations.

ASTM standards for high-strength reinforcing steels have recently been issued and have strength requirements as shown in Table I. The A432 and A16 steels have a required minimum yield point of 60,000 psi, and A431 steel has a minimum yield of 75,000 psi. The A432 and A431 steels are alloy steels, and the A16 steel is a relatively high carbon steel. All three are hot-rolled and have ultimate strengths well in excess of the yield points.

It has been well established that an overall factor of safety against flexural failure of 2.1 is more than adequate for bridges. To approximate this factor in the design of flexural members with varying grades of reinforcing steel, the steel at design loads should have a computed stress of about $\frac{1}{2}$ the yield stress. For A432 and A16 steels the computed design stress should be about 30,000 psi, and for A431 steel it should be about 37,500 psi. This is a sizable increase in stress over the presently allowed 20,000 psi and points to a necessary reconsideration of the four basic design requirements.

TABLE I—ASTM STANDARDS
for
HIGH STRENGTH CONCRETE REINFORCEMENT

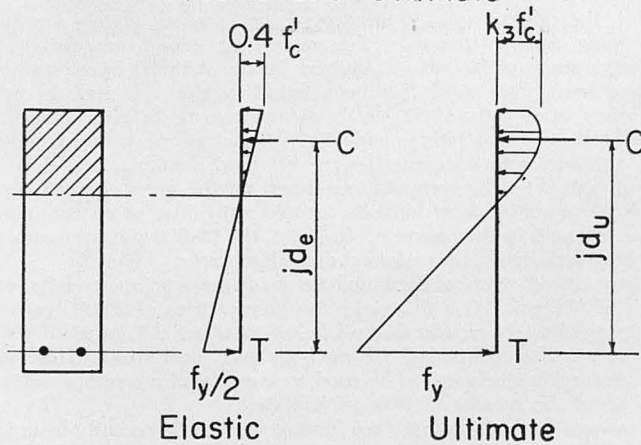
ASTM Designation	Type of steel	Minimum yield point, psi	Minimum ultimate strength, psi
A432-59T	Billet	60,000	90,000
A16-58T	Rail	60,000	90,000
A431-58T	Billet	75,000	100,000

Adequate Safety Against Static Failure

The first of the four requirements, adequate safety against static failure, is relatively easy to insure if the ultimate capacity of the member can be computed. After this capacity is computed, it is reduced by appropriate load factors to obtain the safe service load. Of course, in design, the reverse procedure is followed. The known service loads are multiplied by the desired load factors to determine the required ultimate load. A member is then designed to resist this load.

The stress in a reinforced concrete beam at design load for balanced design is illustrated in Figure 2. For bridges the maximum allowable stress in the con-

STRESS RELATION FOR BALANCED DESIGN
Elastic and Ultimate



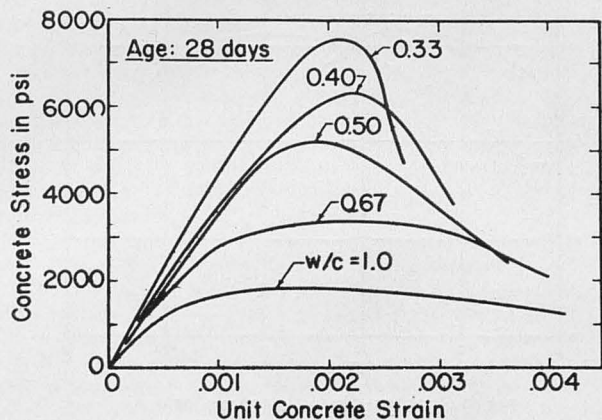
$$\frac{j d_u}{j d_e} = 1.05 \pm$$

Fig. 2

crete is $0.4f'_c$. This stress reduces as we proceed downward toward the tensile steel. A point of zero stress is reached at the neutral axis of the beam. The volume of the stress block is the total compressive force on the concrete. The allowable stress for an intermediate grade steel is 20,000 psi or $\frac{1}{2}$ the minimum yield value of 40,000 psi. The area of the steel multiplied by its stress is the total

tensile force. As load is increased above the design load, the stresses on the concrete and steel increase, and the neutral axis moves slightly upwards when normal amounts of reinforcing steel are used. This condition continues with application of additional load until the steel stress is equal to its yield value. At this point the steel will yield with no increase in stress. As the load further increases, the moment arm "jd" increases until the product of the compressive force and "jd" becomes a maximum. This establishes the ultimate load, and any load beyond this point will cause failure of the beam.

The stress-strain curve of the concrete at ultimate load is a function of the concrete strength, as shown in Figure 3. Because of the variation in these curves,



FLEXURAL STRESS-STRAIN CURVES

Fig. 3

it is rather difficult to represent them exactly by a single mathematical expression. However, the total compressive force and its center of gravity can be easily established by using an equivalent stress block. With the position of resultant compressive force known the moment arm "jd" can be determined. Since the total compressive force must equal the total tensile force, the ultimate capacity can be determined by multiplying the moment arm "jd" by the tensile force in the steel. With the exception of beams with large amounts of steel, the tensile force is equal to the area of steel multiplied by its yield stress.

There is little change in the "jd" value from the elastic design condition to the ultimate condition. From balanced design at elastic load to ultimate load, "jd" increases about 5 per cent and the steel stress doubles. Therefore, the overall factor of safety, the ratio of the ultimate moment to the design moment, is 1.05×2 , or 2.1 as previously stated.

If the tensile stress in the steel remains constant and does not decrease after the steel has yielded, a gradual deflection failure rather than an abrupt failure is assured. The tensile stress in the steel will remain constant after the stress reaches yield if the stress-strain curve of the steel has a definite yield plateau.

Figure 4 shows the stress-strain curves of four steels: a high-yield steel meeting ASTM A431, a 100,000 psi-yield low-alloy steel, a 60,000 psi-yield cold-worked steel, and an intermediate grade steel. The intermediate grade steel is the only curve giving a definite yield plateau of any consequence. The curves of the other steels show a required increase in stress for an increase in strain beyond the

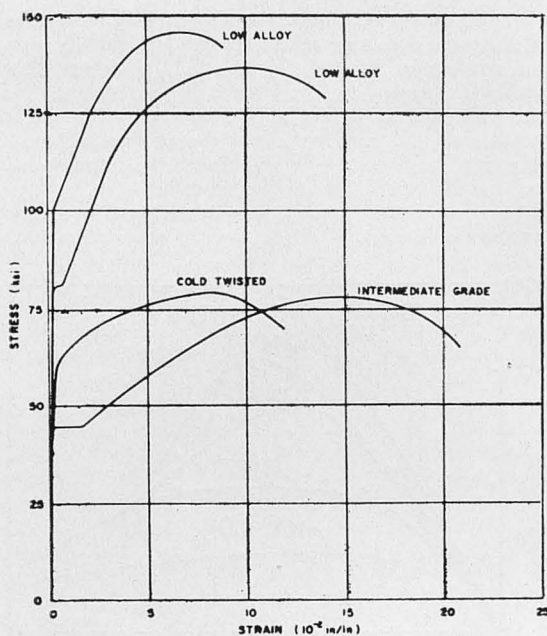


FIG. 10— STRESS-STRAIN DIAGRAMS OF VARIOUS REINFORCING STEELS

Fig. 4

yield point. If normal amounts of high-strength steels are used, the tensile force at ultimate capacity can be conservatively taken as the area of steel multiplied by the yield stress. In an actual case, the stress in high-strength steel at ultimate capacity will more than likely be greater than the yield stress and the true ultimate capacity will be greater than the computed value.

A similar situation to beams in flexure exists in columns which have large bending moments. In columns with large direct loads and relatively small moments, the steel in tension usually will not reach yield stresses. The actual stresses are computed by an analysis of strains which is applicable regardless of the yield strength of the steel. In long columns which buckling may determine the capacity, the same methods will hold for intermediate and high-strength steels.

Adequate Safety Against Fatigue Failure

The second of the four requirements, adequate safety against fatigue failure, is a function of the fatigue properties of the material, the number of applications of load, the change in stress for each load application, and the maximum stress for each load application. The many fatigue tests on prestressed concrete members have proved concrete's resistance to fatigue-loading at high stress levels with maximum change in stress.

Tensile fatigue tests on high-strength steel bars have been conducted both in this country and in Europe. These tests have been highly successful as far as fatigue properties are concerned; however, few tests have been conducted on the bars in flexural members. The Portland Cement Association has begun tests of this type. Although the results have not yet been completely analyzed, it appears that

fatigue will be no problem. In addition to these tests, the AASHO Test Road should give an excellent indication of the fatigue qualities of reinforced concrete members at high stress levels. Two of the reinforced concrete bridges on this test road are designed for 40,000 psi steel stresses. These bridges should be a severe test even though the reinforcing steel used is probably not considered high-strength steel.

Satisfactory Crack Formations

The third of the four basic requirements, satisfactory crack formations promises to be the most difficult to predict. The European Concrete Committee has recently completed extensive studies on the maximum crack widths permissible in respect to appearance and corrosion protection. These studies included a close examination of available laboratory data and the results of inspections and crack measurements made on many existing buildings and bridges. The studies indicated that crack widths varying from 0.004 in. to 0.012 in. are permissible, depending on exposure conditions of the structure or member. In structures exposed to corrosive surroundings, the Committee recommended that widths be held to a maximum of 0.004 in. to 0.008 in. Cracks having a width 0.012 in. are permitted for members in dry climates and members protected in the interior of buildings.

Reviewing these recommendations, it appears that bridges in some parts of the U.S. may need a double requirement for maximum crack widths. Deck surfaces of bridges in cold or severe climates are often heavily salted during the winter months to melt snow and ice. The effect of the salt and freeze-thaw could well be the deciding factor for crack widths over the supports of continuous bridges. On the other hand, larger cracks may be permissible in the semi-protected positive moment sections. Coastal regions, where salt air and salt spray can come in contact with the structure, may also require special consideration.

If it is assumed that the recommendations of the European Concrete Committee are valid and acceptable, it would not be too difficult to assign maximum allowable crack widths to the structures in this country. However, the important problem of predicting the crack widths still remains.

The formation of cracks is influenced by several factors. The most important are: Steel stress, bar diameter, type of bar deformation, surface condition of the bars (whether clean or rusty), shape of the concrete beam, distribution of the bars in the tensile portion of the beam, bar cover, and position and number of stirrups. Most of these factors are understood, although work still needs to be done on methods of predicting crack widths.

Many formulas have been written from experimental data to predict average crack widths, maximum crack widths and crack spacing. Nearly all of these formulas are related to test data on European bars, which have poor bond characteristics by comparison with our ASTM A305 bars. For this reason, these formulas are likely to be overly conservative for the present ASTM high-strength steel bars.

Nearly all the expressions written so far are in the form of the formula for average crack width proposed by the European Concrete Committee:

$$W = K \frac{Df_s}{P_s} \text{ in mm.}$$

Where

D = bar diameter in mm

f_s = steel stress in Kg/mm²
steel area

P_s = $\frac{\text{steel area}}{\text{effective concrete area}}$

K = 0.95×10^{-3} for deformed bars

Research in the U.S. indicates that this formula gives good results for the old type deformed bars, but may tend to overemphasize the bar diameter for the A305 bars.

A similar type formula has been presented by Clark from tests on intermediate grade steel at high stresses. Clark's formula is somewhat more refined than the one above and is as follows:

$$W = 2.27 \times 10^{-8} \frac{h-d}{d} \frac{D}{p} \left[f_s - 56.6 \left(\frac{1}{p} + N \right) \right]$$

Where

- W = average width of cracks, in.
- D = diameter of reinforcing bar, in.
- f_s = computed stress in reinforcement, psi
- h = overall depth of beam, in.
- p = ratio of longitudinal reinforcement
- d = depth to the longitudinal reinforcement

This formula may also overemphasize the diameter of the reinforcing steel bars and the effect of the ratio of steel to the total concrete area.

The formulas for average crack width still need to be modified in order to determine the maximum width. The ratio of the maximum crack width to the average crack width apparently has a rather large variation. Reported ratios have been as high as 2.77 and as low as 1.18. These variations are for different types of beams.

The calculation of maximum crack width has been compulsory in Sweden since 1947 for all highway bridges. The formula used is one developed by Jonsson, Osterman and Wastlund, and for pure bending is as follows:

$$W_{\max} = KD \left[\frac{I_c}{dA_s e_t} \times \frac{f_s}{E_s} \right]^{2/3}$$

Where

- W_{\max} = calculated maximum crack width, in.
- K = 0.23 for plain bars
- K = 0.16 for KAM 40
- D = diameter of bar, in.
- d = depth from compression face to centroid of reinforcement steel, in.
- A_s = area of steel, sq. in.
- f_s = stress in steel, psi
- E_s = modulus of elasticity of steel, psi
- I_c = moment of inertia of gross concrete section
- e_t = distance from neutral axis to extreme fiber on tension face

In general it is expected that these formulas can be used to give conservative answers if good design and detailing practices are followed.

The portion of the beam section in tension should be made as narrow as practical and still maintain clearance for the reinforcing steel. Clearance between the steel bars should be a minimum, providing only the space necessary for adequate placing of the concrete. Beam webs should also be relatively thin in order to reduce the effect of secondary stresses on crack widths. If calculations of crack widths indicate they may be too wide, additional nominal longitudinal steel near the vertical surfaces of the webs will aid in controlling these widths. Clearance from the face of the concrete to the main reinforcing steel should also be kept to a minimum.

Best crack control will be obtained if the tension area of the concrete is designed so that all excess concrete is reduced to a minimum.

With these factors in mind it appears that thin-webbed members will provide structures with minimum crack widths. Box girders, I-shaped beams, and thin-webbed T-beams are examples of this type member. When flanged members, box girders and I-beams are used, the tension steel should be spread throughout the full flange width.

Satisfactory Deflections

The fourth basic requirement, satisfactory deflections, can be assured if full recognition is given to the phenomenon of creep. This also applies to concrete reinforced with intermediate-grade steel. Here the usual practice is to compute initial deflections on the basis of the gross concrete section using the modulus of elasticity of the concrete at the time the load is applied. For long-term loads a reduced modulus of elasticity is used to account for creep.

Investigations on high-strength steels point to the use of the transformed section, or cracked section, for computing all deflections. Deflections for long-term loadings should still be based on a reduced modulus of elasticity of the concrete. Actually, the reduced modulus can be taken in account by multiplying the computed initial deflection by a ratio of the instantaneous modulus to the reduced modulus. In average climates this ratio can be taken as 3. In damp climates a value should be chosen between about 2.5 and 3, and for very dry climates a value between 3 and 3.5 should give good results.

Safety Factors

As previously stated, the maximum allowable steel stress of 20,000 psi for reinforced concrete structures has served as a guarantee of fulfilling the four basic requirements. It appears that the use of high-strength steels will require a more detailed examination of these requirements. Therefore, it seems logical to reconsider the load factors so that more realistic and economical designs can be obtained.

The present AASHO Specifications for prestressed concrete require a design for ultimate load as expressed by Equation 1:

$$\text{Ultimate Load} = 1.5D + 2.5L \dots\dots\dots (1)$$

where

- L = live load plus impact
- D = live load plus impact

Since the overall factor of safety is equal to the ultimate load divided by the service load, Equation 1 can be rewritten for comparison with 2.1 now required for reinforced concrete bridges.

$$\text{Overall factor of safety} = \frac{1.5D + 2.5L}{D + L} \dots\dots\dots (2)$$

Equation 2 can be evaluated by selecting ratios of dead load to live load and substituting in the equation the value of the dead load in terms of the live load.

If the dead load is known to equal the assumed design value, the live-load factors of safety can be determined in a similar manner. The equation:

$$\text{Live-load factor of safety} = \frac{(1.5D + 2.5L) - D}{L} \dots\dots\dots (3)$$

is used and the live load is substituted in terms of the dead load for selected dead to live-load ratios.

Equations 2 and 3 have been evaluated and plotted in Figure 5 for dead to live-load ratios from 0 to 6. The dashed line representing the factor of 2.1 has

also been plotted for comparison. At $D/L = 0$, the overall factor and the live-load factor for $1.5D + 2.5L$ are both equal to 2.5. As $D/L = 1.7$, the overall factor of safety is equal to the 2.1 for elastic design. When $D/L = 6$, the overall factor has reduced to 1.64, but the live-load factor has increased to 5.5. The reduction of the overall factor of safety with increase in D/L simply means that recognition is being given to the fact that the dead loads are relatively constant, causing little or no change in stress throughout the life of the structure. Even with this recognition, the live-load safety factor is increasing at a rapid rate, insuring adequate resistance for possible large future overloads.

A similar situation is allowed by AASHO Specifications for the design of culverts. The specification states that "in order to have the effect of increasing the allowable design load stresses 40 per cent more than allowed for live load, the effective weight of earth backfill may be taken as 70 per cent of its actual weight."

Expressing the specification mathematically leads to the following:

$$\text{Design load} = 0.7D + L \dots\dots\dots (4)$$

The ultimate load using the culvert design can now be solved by multiplying the design load, Equation 4, by 2.1.

$$\text{Ultimate load} = 1.5D + 2.1L \dots\dots\dots (5)$$

The overall factor of safety can also be determined by dividing Equation 5 by the service load.

$$\text{Overall factor of safety} = \frac{1.5D + 2.1L}{D + L} \dots\dots\dots (6)$$

And the live-load factor of safety is given by Equation 7.

$$\text{Live-load factor of safety} = \frac{(1.5D + 2.1L) - D}{L} \dots\dots\dots (7)$$

Equations 6 and 7 have been evaluated for D/L ratios from 0 to 6 and superimposed on the curves of Figure 5, as shown in Figure 6. The overall factor of safety and the live-load factor of safety begin at the identical point as the elastic design for bridges. As D/L increases, the overall factor approaches the value for the prestressed requirements. The live-load factor is somewhat less by a constant amount, though still quite high.

All of these factors of safety are used in the present AASHO Specifications and have proven satisfactory over a relatively long period of time. The value of 2.1 for reinforced concrete structures other than culverts seems excessive for D/L ratios greater than about 1.5. In view of the experience with load factors used in prestressed concrete and in culvert design, it would seem reasonable to reconsider the load factors required for reinforced concrete bridges.

Research

Research on crack formation is now underway at Portland Cement Association Laboratories and at Cornell University. Most of the factors affecting crack widths and spacing are now known and understood. It is therefore expected that the research now underway will provide the additional data needed to establish reasonable design methods for predicting widths and spacing.

To augment the Laboratory research, a test bridge using high-strength steel is under design by the New York Department of Public Works. This bridge is expected to be constructed during 1961. The 2 end spans will be 35 ft. and the

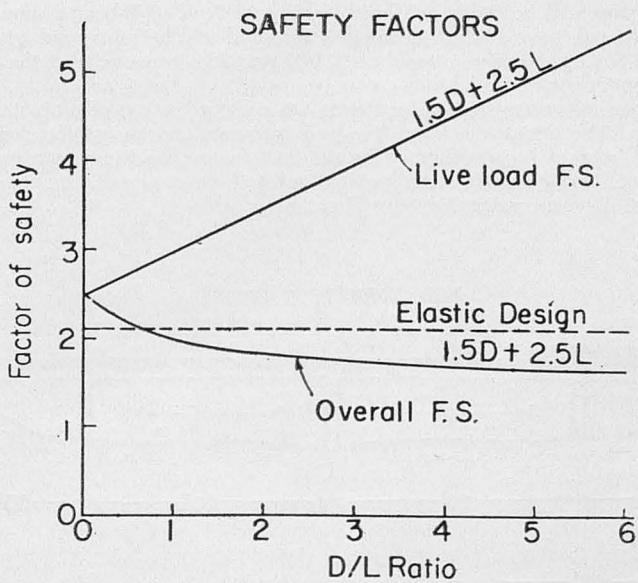


Fig. 5

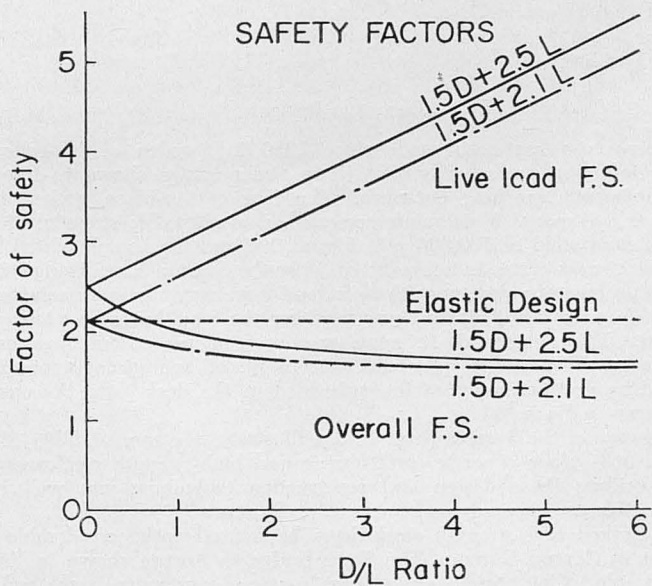


Fig. 6

2 center spans will be 60 ft., as shown in Figure 7. The T-beams shown in the cross-section will have a width of about 10 in. and will be reinforced with ASTM A432 steel having a minimum yield of 60,000 psi. It is expected that the structure will be continuous.

The general criteria set up for the design call for the use of ultimate strength procedures. The design dead-and live-load moments and shears for each section along the girders will be multiplied by the load factors used in prestressed bridge design. Each section will be designed to resist these required ultimate moments and shears. Moment redistribution will not be considered.

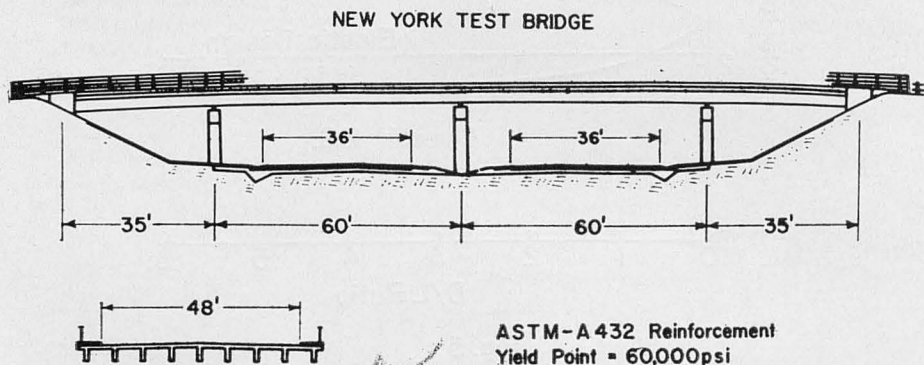


Fig. 7

European Bridges

Extensive use has been made since 1950 in Sweden of hot-rolled high-strength reinforcing steels for bridges. The girder bridge shown in Figure 8 is across the Sagan river near Ostanbro. The four cast-in-place girders are continuous over five spans. Main reinforcement has an allowable stress of 43,000 psi and a minimum yield of 100,000 psi. Figure 9 shows the complete structure.

All the girders in the 12 approach spans of the Svartan River Bridge shown in Figure 10 are factory precast with deformed bars used as main reinforcement. An allowable stress of 43,000 psi was used for the steel having a yield point of 86,000 psi. The center 131-ft. span was cast in place and prestressed by posttensioning. The bridge deck was cast in place, and girder continuity was established by posttensioning cables embedded in the deck slab. A construction view is shown in Figure 11.

The design of the Svartan River Bridge illustrates the compatibility of precast concrete, cast-in-place concrete, prestressing and high-strength reinforcing steels. In many bridges these design and construction techniques can well be used together to increase economy and improve performance.

Cold-worked high-strength steels have been used with considerable success for bridges in Central Europe. The Drau bridge in Austria shown in Figure 12 is an example. This bridge is a cast-in-place continuous slab and girder structure with 150-ft. end spans and a 226-ft. center span. An allowable design stress of 31,000 psi was used in the steel with a specified yield strength of 57,000 psi.

Conclusion

The use of high-strength steels in bridge construction will require a reconsideration of the four basic requirements a structure should fulfill. Three of these requirements can be adequately evaluated by present design methods. The fourth, satisfactory crack formation, is the subject of present field and laboratory research from which early answers are expected.

When these answers are available, the use of high-strength steel with consequent reduction in steel cost per unit of load-carrying capacity, reduction in concrete quantities, and reduction in dead load should in many instances result in more economical reinforced concrete bridges.

References

1. HOGNESTAD, E., and WINTER, G., "High Strength Reinforcing Steels for Concrete Bridges." Proceedings of the Thirty-Ninth Annual Meeting of the Highway Research Board, Jan. 1960, Vol. 39.
2. GURALNICK, S. A., "An Investigation of High Strength Deformed Steel Bars for Concrete Reinforcement." Journal of the American Concrete Institute, Sept. 1960, Proceedings, Vol. 57.
3. YU, W. W., and WINTER, G., "Instantaneous and Long-Time Deflections of Reinforced Concrete Beams under Working Loads." Journal of the American Concrete Institute, July, 1960, Proceedings, Vol. 57.
4. MATHEY, R. G. and WATSTEIN, D., "Effect of Tensile Properties of Reinforcement on the Flexural Characteristics of Beams." Journal of the American Concrete Institute, June 1960, Proceedings, Vol. 56.
5. WASTLUND, G., "Use of High-Strength Steel in Reinforced Concrete." Journal of the American Concrete Institute, June, 1959, Proceedings, Vol. 55, pp. 1237-1250.
6. GASTON, J. R., and HOGNESTAD, E., "Precast Concrete Girders Reinforced with High Strength Deformed Bars." Portland Cement Association Development Department Bulletin D24. Reprinted from Journal of the American Concrete Institute, October, 1958, Proceedings, Vol. 55, pp. 469-484.
7. CLARK, A. P., "Cracking in Reinforced Concrete Flexural Members." Journal of the American Concrete Institute, April, 1956, Proceedings, Vol. 52.
8. EFSEN, A. and KRENCHHEL, H., "Tensile Cracks in Reinforced Concrete." Research Laboratory of Building Technique, Technical University of Denmark, Bulletin No. 9, Copenhagen, 1959.
9. HOGNESTAD, E., HANSON, N. W., and McHENRY, D., "Concrete Stress Distribution in Ultimate Strength Design." Portland Cement Association Development Department Bulletin D6. Reprinted from Journal of the American Concrete Institute, December 1955, Proceedings, Vol. 52, pp. 455-479.