Objectives and Early Research of Autostress Design

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The early research of autostress design started with the simple idea of using plastic design of continuous members so that prismatic members could be used from one abutment to the other. This was expected to result in significant savings in fabrication costs. Because most steel bridge beams and girders do not satisfy the compact section requirement of conventional plastic design, a new approach based on the concept of an effective plastic moment made it possible to adapt plastic design for bridge design. Serviceability requirements received special attention because prismatic members designed for strength could very well yield under actual loading conditions such as the overloads specified by the American Association of State Highway and Transportation Officials. Procedures were developed to account for the effects of local yielding. Advantage was taken of the ability of steel members to redistribute automatically any excess moments; hence the term autostress design.

Research on autostress design of highway bridges started with a small design project of a rolled beam bridge previously designed by the load factor design (LFD) method (*1*). The original design consisted of continuous rolled beams over two spans of 70 ft each with cover plates welded to the beams in the region near the intermediate support. The objective was to eliminate the cover plates, which would simplify fabrication and eliminate the undesirable fatigue detail at the termination of the cover plate. Because LFD is based on elastic analysis, it would have required the use of a heavier beam if the cover plates were eliminated. An alternative method would have been to use a pin-hanger connection near the intermediate support to balance the positive and negative moments so that no increase in weight would be needed. It was clear that some type of inelastic design procedure would be necessary to minimize the total of the material and fabrication costs.

First, an attempt was made to use classical plastic design as currently included in the American Institute of Steel Construction (AISC) specification (*2*). However, it soon became apparent that it would be difficult to meet the compact section requirements of plastic design for beams in bridge construction. Research eventually solved this problem by introducing the concept of an effective plastic moment.

The second roadblock that was encountered involved serviceability under overload conditions. Plastic design, even on the basis of the effective plastic moment concept, indicates a very large load-carrying capacity. Thus, serviceability under overload conditions must be considered. To guard against excessive permanent deflections, the American Association of State Highway and Transportation Officials (AASHTO) Specifications (*3*) include the provision that the maximum stress in a composite beam under overload conditions not exceed 95 percent of the yield stress. For noncomposite beams, the limit is 80 percent. If these limits are applied to the negative bending sections of continuous bridge beams at intermediate supports, the advantages of plastic design for maximum load would largely be negated. Thus, the second part of the research addressed the problem of how to deal with the effects of yielding at intermediate supports. Because continuous steel beams automatically adjust for the effects of local yielding through redistribution moments, the term autostress design was introduced to identify the proposed procedures.

The overall objective of the project was to design continuous beam and girder bridges with steel members of constant cross section along their entire length, or at least within field sections. The research established (a) methods for calculating strength at maximum load conditions on the basis of plastic design with effective plastic moments and (b) procedures to account for the effect of local yielding at overload conditions. The work was summarized in an American Iron and Steel Institute (AISI) bulletin (*4*) and implemented in the AASHTO Guide Specifications (*5*).

The highlights of the research are described in this paper. The work was performed as part of Project 188 sponsored by the AISI.

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PLASTIC ROTATION CAPACITY OF BRIDGE BEAMS

An important part of Project 188 was the investigation of the plastic rotation capacity of composite sections in negative bending as it occurs near intermediate supports at maximum load. The cross section is shown in Figure 1. When the section is subjected to negative bending, the reinforcing steel is assumed to act compositely with the beam so that the neutral axis shifts toward the slab. Yielding further shifts the neutral axis to a location, *DWCP* from the bottom flange, which is in compression. For adequate plastic rotation capacity at the first hinge to form, the plastic design procedures of Chapter N in the AISC Specification (*2*) require that the width-to-thickness ratio of the compression flange not exceed the value $BF/(2 * TF) = 7$ (where *BF* equals width of compression flange and *TF* equals thickness of compression flange) for steels with yield points of 50 ksi. Similarly, the section-depth over web-thickness ratio must not exceed $412/FY^{1/2}$. For the section shown in Figure 1, it is assumed that $2 * DWCP$ corresponds to 95 percent of the depth of a symmetrical section.

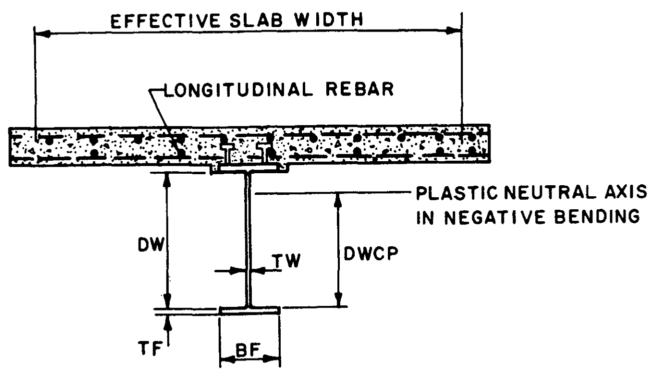

FIGURE 1 **Cross section at interior pier.**

The AISC requirement can then be restated as

$$\frac{2 * DWCP}{TW} \leq \frac{0.95 * 412}{FY^{1/2}} \tag{1}$$

where

$DWCP$ = depth of web in compression for plastic stress distribution,
TW = thickness of web, and
FY = yield point of steel.

For steel with a yield point of 50 ksi, this requirement becomes

$$\frac{2 * DWCP}{TW} \leq 55 \tag{2}$$

In the international literature, sections that meet these requirements are designated as Class 1 sections (*6*).

The AISC Specification (*2*) has less severe requirements when sections are only required to just reach the plastic moment. Such sections are known as Class 2 sections in international usage (*6*). The width-to-thickness ratios are

$$\frac{BF}{2 * TF} = \frac{65}{FY^{1/2}} \tag{3}$$

for compression flanges, and

$$\frac{2 * DWCP}{TW} = \frac{0.95 * 640}{FY^{1/2}} = \frac{608}{FY^{1/2}} \tag{4}$$

for webs. These ratios are 9.2 and 86.0 for steels with a yield point of 50 ksi. Equation 4 is again restated to be applicable to unsymmetrical sections. The AISC Load and Resistance Factor Design Specification (*7*) further liberalizes Equation 4 by using the distance between flanges rather than the depth of section. This would remove the factor 0.95 in Equation 4. One of the objectives of the early research was to develop a procedure that would enable Class 2 sections to be used in plastic design. The approach was to establish an effective plastic moment level at which Class 2 sections would have sufficient plastic rotation for mechanism formation. Class 2 sections then could be treated as Class 1 sections in plastic design but with a reduced effective plastic moment. The effective plastic moment of a Class 2 section is defined as the level on the unloading portion of the moment-rotation curve where the plastic rotation is the same as that at the plastic moment level for a Class 1 section. The following procedure for determining the effective plastic moment was proposed by Haaijer et al. (*8*).

PROCEDURE FOR CALCULATING EFFECTIVE PLASTIC MOMENT

The terms that describe the geometry of a composite steel-concrete T-section are defined in Figure 1. The section is assumed to be in negative bending corresponding to the condition near intermediate supports of continuous beams. The AISC rules for Class 1 compact sections in plastic design were used as the basis for the following development. According to AISC, the compression flange is considered compact (Class 1) for plastic design at a yield point of 50 ksi if the flange slenderness, $BF/(2 * TF)$, is equal to or smaller than 7.0. For other yield points, the requirement is approximately

$$\frac{BF}{2 * TF} \leq 7.0 * \left(\frac{50}{FYF}\right)^{1/2} = \frac{49.5}{FYF^{1/2}} \tag{5}$$

where FYF equals yield point of compression-flange steel. Here FYF equals the specified minimum yield point of the flange material in ksi. Rather than limit the maximum value of the flange slenderness according to the specified minimum yield point, an effective yield point of the flange material $FYFE$ is defined according to the actual flange slenderness so that

$$FYFE = 9{,}800 * \left(\frac{TF}{BF}\right)^2 \leq FYF \tag{6}$$

where $FYFE$ equals effective yield point of compression-flange steel. For this effective yield point, the flange can be considered Class 1 compact.

Because AISC gives web-slenderness ratios only for symmetrical sections, a few adjustments are needed to account for the shift of the neutral axis in a composite section. Defined here is the effective web slenderness at the plastic moment of the composite section shown in Figure 1 as $2 * DWCP/TW$; for a symmetrical shape, $2 * DWCP = DW$. The AISC requirement is based on the overall depth of the section. The actual depth of the web in between the flanges is about 95 percent of the overall depth for most rolled beams. The AISC requirement can then be restated as

$$\frac{2 * DWCP}{TW} \leq \frac{0.95 * 412}{(FYW)^{1/2}} \tag{7}$$

where FYW equals yield point of web steel. Now replace the yield point of the web material, FYW, by the effective yield point of the web, to obtain

$$FYWE = 38{,}300 * \left(\frac{TW}{DWCP}\right)^2 \leq FYF \tag{8}$$

where $FYWE$ equals effective yield point of web steel.

FYFE and *FYWE* are limited to *FYF* because flange and web buckling are controlled primarily by flange strain. Finally, normalize both *FYFE* and *FYFW*, with respect to the actual corresponding yield stress, and obtain the reduction factors $RF = FYFE/FYF$ and $RW = FYWE/FYF$ (where *RF* equals reduction factor for plastic moment of flange and *RW* equals reduction factor for plastic moment of web) for the compressed flange and web, respectively. The negative plastic moment, *MPN*, of the section is the sum of the contribution of the flanges and web, so that

$$MPN = MPNF + MPNW \tag{9}$$

where

MPN = negative plastic moment,
$MPNF$ = negative plastic moment of flanges (including rebars), and
$MPNW$ = negative plastic moment of web.

The effective plastic moment *MPNE* is then obtained by multiplying the components of *MPN* by the respective reduction factors

$$MPNE = RF * MPNF + RW * MPNW \tag{10}$$

where *MPNE* equals effective negative moments. *MPNE* can then be used in plastic design with Class 2 section in lieu of *MPN* in conventional plastic design with Class 1 compact sections.

SERVICEABILITY AT OVERLOAD

The preceding analysis and design procedures ensure that the steel beams or girders have adequate strength to resist the fully factored design loads. In addition, bridge structures must show satisfactory service performance. One important requirement is that beams and girders have limited permanent deflections when subjected to occasional overloads during the life of the structure. The current LFD specifications assume that the overload will not exceed ⅔ times the service load. Because the specified service load according to AASHTO consists of one HS20-44 truck in each lane, the overload can be considered as one HS33 service load in each lane (*9*). In reality, this requirement was derived from the loading condition of two HS20 trucks piggybacked in one lane (*10*). To minimize permanent deflection under this loading condition, AASHTO limits the maximum stress in the steel to 95 percent of the yield stress for composite beams and girder and to 80 percent for noncomposite members. These requirements were derived from tests of simple span bridges. However, many designers also apply these limits to cross sections of continuous members at intermediate supports. This requirement would negate the advantage achieved from the previously described plastic design procedures (*9*).

The crux of the autostress project therefore became the proposition that sections in negative bending at supports would be permitted to yield under overload (HS33 service load). As shown in Figures 2 and 3, this local yielding caused the beam or girder to deform plastically. Illustrated in Figure 2 is the deformed shape of a two-span continuous bridge beam after the overload has caused local yielding near the center support. The loads shown in Figure 2 include dead load plus ⅔ times the AASHTO lane load, including impact, which usually creates the critical negative bending moment at the center support. In the absence of dead load or tie down, the inelastic rotation resulting from the local yielding would cause the

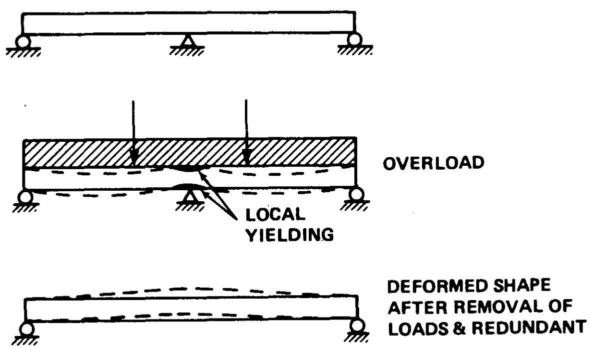

FIGURE 2 Deformations caused by overloads.

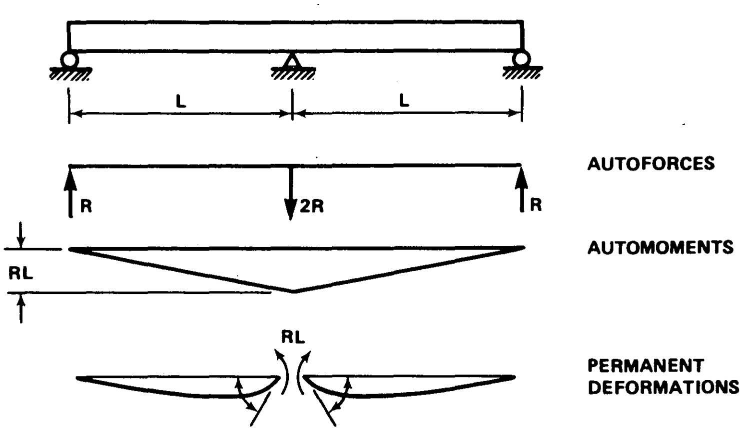

FIGURE 3 Automoment and permanent deformations.

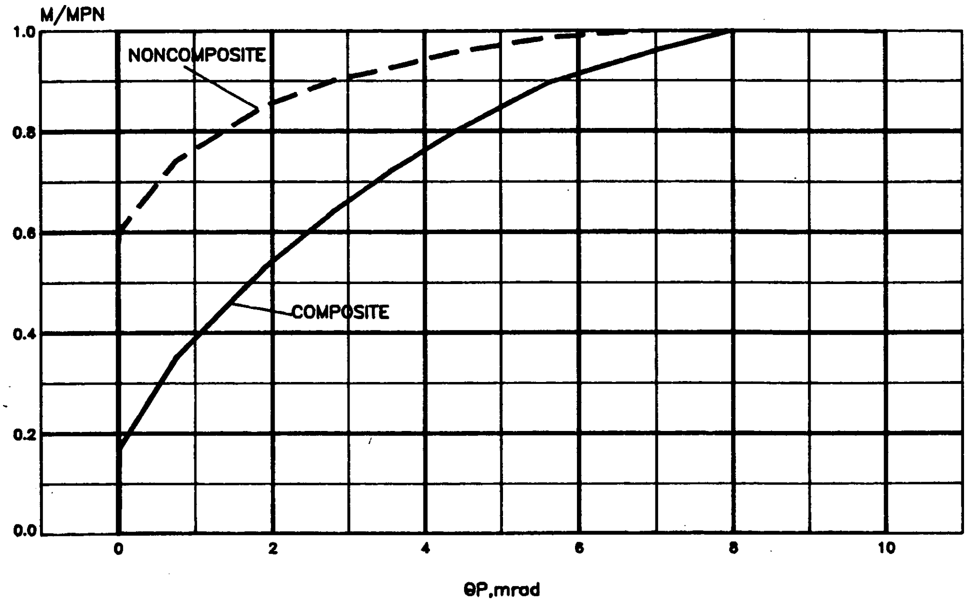

FIGURE 4 Moment versus plastic rotation curves for design.

beam to lift off the center support. The forces shown in Figure 3 prevent this movement and create the indicated automoments. The resulting permanent deformations, including both the elastic and inelastic contributions, are shown in Figure 3. The automoments ensure that the structure remains elastic when subjected to loads not exceeding the initial overload. They do not affect the stress ranges related to fatigue design.

The particular automoment diagram shown in Figure 3 could, of course, be achieved without yielding the structure by the well-known procedure of constructing the girder with the center support in an elevated position and lowering the finished structure to its final position. The advantage of autostress design is that it achieves the same result without the cost of manipulating the completed structure.

To establish the moment-rotation characteristics of bridge cross sections at intermediate supports, an extensive number of tests were conducted (*11–13*). The two curves shown in Figure 4 were derived from the experimental curves for purposes of design. The two curves make it possible to determine the actual automoments and the corresponding permanent displacements at overload for noncomposite and composite sections, respectively, so that appropriate camber requirements can be established (actual curves could, of course, be used). The curves apply to shapes that can reach or exceed *MPN*. For shapes that can only reach the yield moment in negative bending, *MYN*, the ordinate denominator could simply be changed from *MPN* to *MYN*. However, the current Autostress Guide Specification (*5*) restricts the use of the two curves to Class 2 compact sections.

The procedure adopted by AASHTO for determining the automoment and the related inelastic rotation at overload is shown in Figure 5 for a symmetrical two-span bridge member. The pier moment is plotted on the ordinate and the inelastic rotation is plotted on the abscissa. The experimentally based

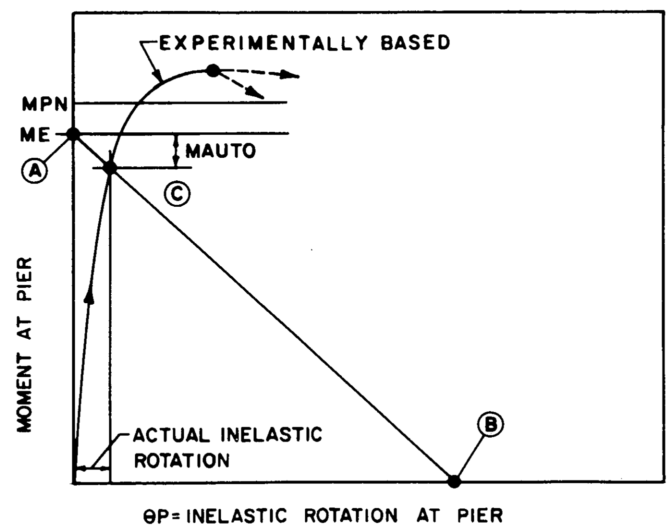

FIGURE 5 Determination of automoment and inelastic rotation.

moment versus inelastic rotation curve is a function only of the pier section and pier-moment gradient. The moment rotation curve is intersected by a straight line connecting Points *A* and *B*. Point *A* is the sum, *ME*, of the moments caused by dead load and overload live load computed from an elastic analysis. Because Point *A* is based on elastic behavior, θP is zero. Point *B* is the total change in slope for two simple beams. This is caused by the dead load and overload live load on the bridge member when the interior pier is assumed to have a free hinge so that the pier moment is zero. The rest of the structure is elastic with the same stiffness properties as those used in computing Point A. From superposition it follows that any combination of pier moment and rotation falls on a straight line between Points A and B. The pier automoment $MAUTO$ and permanent inelastic rotation are located at Point C, the intersection of the straight line and the experimental moment rotation curve. The actual pier moment equals ME plus $MAUTO$. In this example, ME is negative and $MAUTO$ is positive. When a bridge is continuous over more than one pier, carryover of automoments occurs. This may require an iterative solution.

Once the automoments are determined, the corresponding permanent deflections can be determined from an elastic analysis, as shown in Figure 3. In practice, these deflections are added to the dead load deflections in establishing camber requirement.

FIELD TESTING

Before AASHTO approved the 1986 Guide Specification, a load test of a bridge designed by the autostress method was conducted by Roeder and Eltvik (*14*). The bridge was built across the Whitechuck River in the Mt. Baker National Forest near Darrington, Washington. The elevation of this three-span continuous bridge is shown in Figure 6. The bridge was designed jointly by AISI as part of Project 188 and by the Office of Western Bridge Design of the Federal Highway Administration (FHWA) in Denver, Colorado. The bridge was designed for HS20 Service Loads. However, for the overload condition, a special forestry yarding vehicle with a total weight of 210,000 lb was specified. The cross section of the bridge is shown in Figure 7. The simplicity of the steel members is illustrated in Figure 8. Note the connecting plates for cross bracing at 5 ft 6 in. from the pier to meet the lateral bracing requirements of the compression flange for plastic design at maximum load. The design of the center span in between the field splices was governed by the serviceability criterion at overload that the maximum stress in the steel beam should not exceed 95 percent of the yield point after the creation of the automoment and with the overload vehicle in position for maximum moment. This requirement was satisfied with a W30 × 191 A588 steel shape. For the rest of the bridge, a W30 × 116 A588 steel shape was adequate. The calculated deflections caused by dead load on the automoment are shown in Figure 9. The camber requirements were specified as the sum of the two deflection curves.

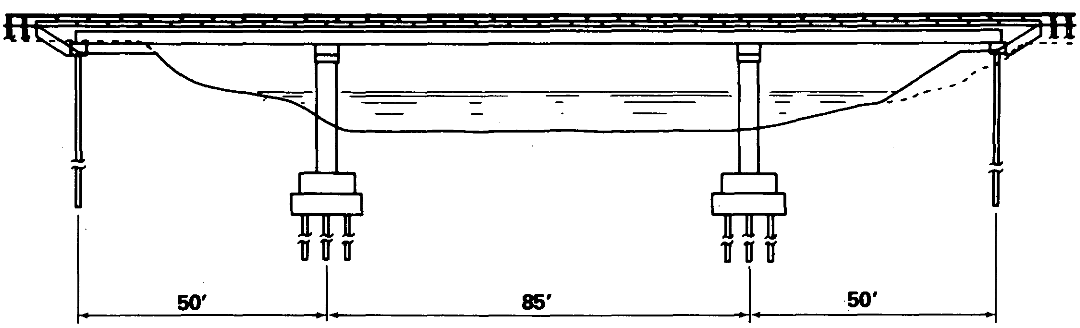

FIGURE 6 Elevation of Whitechuck River Bridge.

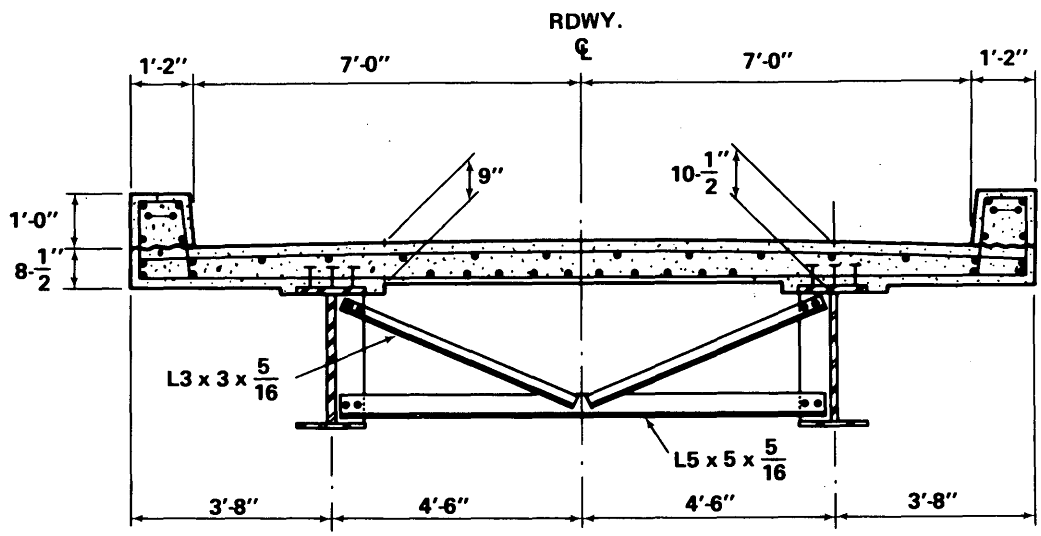

FIGURE 7 Cross section of Whitechuck River Bridge.

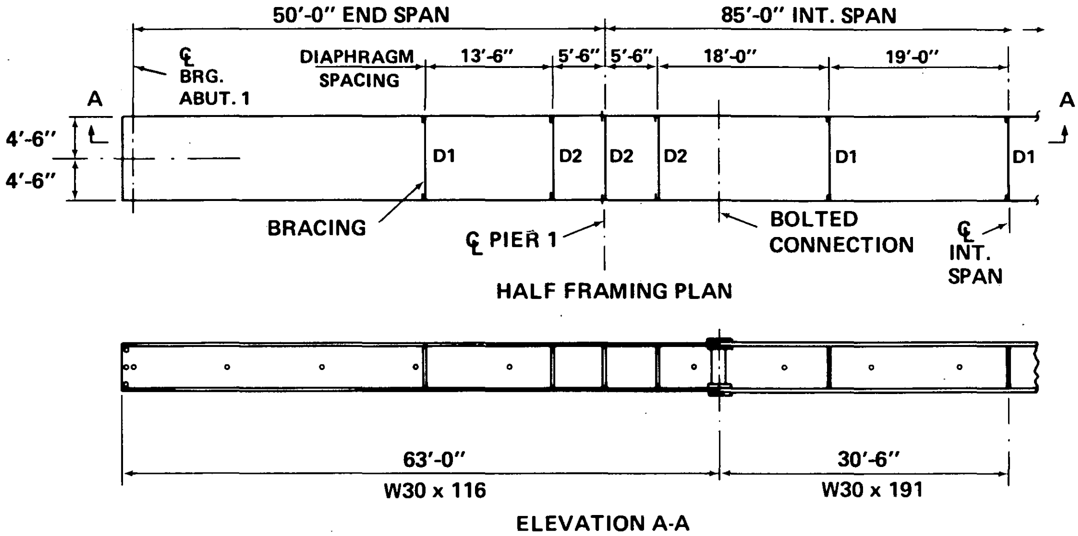

FIGURE 8 **Framing plan and beam elevation of Whitechuck River Bridge (shear connectors not shown).**

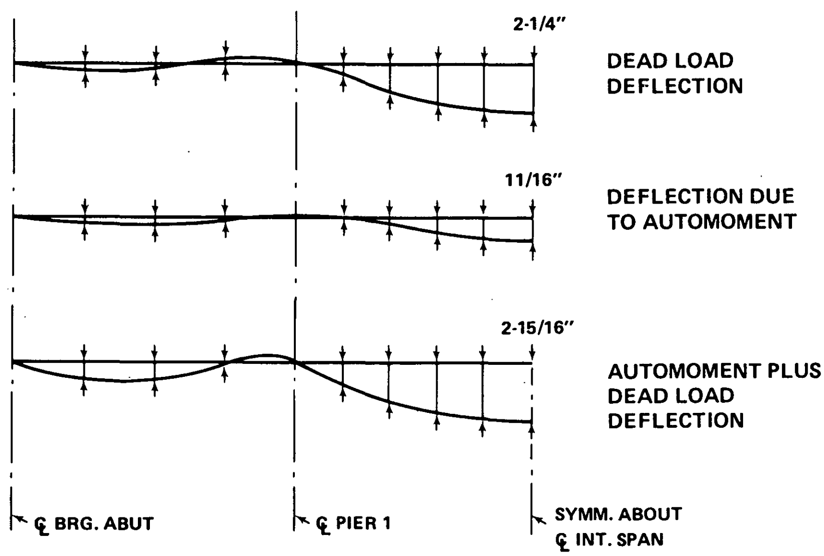

FIGURE 9 **Computed deflection diagrams for Whitechuck River Bridge.**

The field test provided a wealth of information on the structural behavior of the bridge. The investigators concluded that significant yielding took place in the regions of negative bending. Because the bridge was not shored during construction, yielding did occur during placement of the concrete deck. The fabricator also noted some yielding during heat cambering of the beams. The permanent deflections were close to the predictions but occurred in a different sequence than those anticipated in the design. This is a good illustration of the automatic adjustment of the structure when conditions are different from design assumptions. Cracking of the deck was much less than in the laboratory model test, because unshored construction reduces the stresses in the deck. Furthermore, the concrete strength was 2 to 2.5 times the specified value of 3,500 psi after 28 days. Most significantly, deflections stabilized after repeated load applications. Strains and deflections after the seventh load application were elastic.

CONCLUSIONS AND FUTURE WORK

The early research on autostress design led to the successful development of an AASHTO Guide Specification. Several highway bridges designed by this method have already been built. The performance of these bridges demonstrates the viability of the concept and has improved the economy of short-span steel bridges through simplicity of fabrication.

Significant progress has been made toward extending autostress design of Class 3 noncompact plate girders for which the strength is determined by first yielding of the flanges. A model bridge with Class 3 sections has already been tested at the FHWA Turner-Fairbanks Laboratory. Knight et al. (*15*) described the design of the prototype bridge shown in Figure 10.

The bridge has two equal spans of 140 ft. The bottom flange of each of the three girders is constant (1-⅜ in. × 20 in.) along the entire length of the bridge except for 28 ft near the end supports, where the thickness is reduced to ¾ in. The top flange is constant (⅝ in. × 14 in.) along the entire length. Any size section could have been chosen at the pier because the required automoment will be created automatically by whatever yielding takes place at overload. The prototype bridge probably represents the ultimate of what can be achieved because the compression flange at the support is the same size as the tension flange at the point of maximum positive moment. Even for this extreme design, the model bridge performed admirably (*16*).

Work must now be started to develop a design specification for Class 3 plate girders. Schilling (*17*) has proposed a unified autostress design method that will greatly simplify the current design procedures. Also, the proposed load and resistance factor design (LRFD) Specification for Bridges provides an opportunity to review the serviceability requirements. The current overload design checks in LFD should probably be replaced by similar provisions of a service load in LRFD because the service load model represents actual maximum loads anticipated during the life of the bridge.

Inelastic design of steel bridges is also being considered in Europe. Wargsjo and Johansson (*18*) reported on the rotation capacity and post-buckling capacity of steel girders with slender webs. Their test data will also be valuable in extending autostress procedures to girders with slender webs.

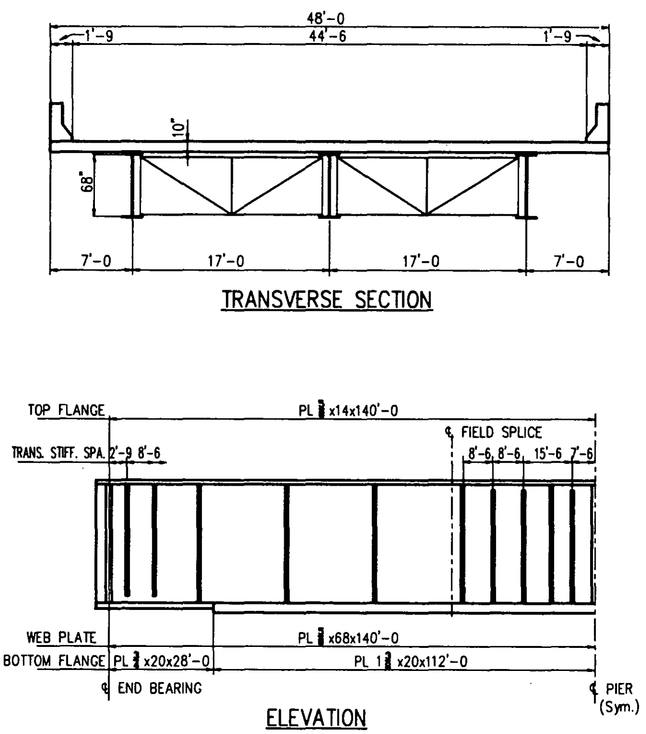

FIGURE 10 Prototype of model bridge tested at FHWA Turner-Fairbanks Laboratory.

A summary of other European tests was published by Johnson and Chen (*19*). The results confirm that composite Class 2 compact sections have significant plastic redistribution capability, as required by the proposed Eurocode 4 for the design of composite members. When uncracked flexural stiffnesses are used for calculating the elastic moments, the draft Eurocode 4 permits up to 30 percent redistribution for Class 2 compact beams at the ultimate limit state. However, Johnson and Chen do not indicate whether a serviceability check is required. The latter often governs the design using autostress procedures, according to AASHTO.

It is encouraging that inelastic procedures appear to gain momentum. This will result in a better use of the ductility of steel and discourage elastic design procedures that result in undesirable details, such as cover plates, that reduce fatigue life.

REFERENCES

1. P. S. Carskaddan. *Autostress Design of Highway Bridges, Phase 1: Design Procedure and Example Design.* Project 188, American Iron and Steel Institute, Washington, D.C., March 8, 1976.
2. *Specification for Structural Steel Buildings - Allowable Stress Design and Plastic Design.* American Institute of Steel Construction, Chicago, Ill., June 1, 1989.
3. *Standard Specification for Highway Bridges*, 14th ed., American Association of State Highway and Transportation Officials, Washington, D.C., 1989.
4. G. Haaijer, P. S. Carskaddan, and M. A. Grubb. *Suggested Autostress Procedures for Load Factor Design of Steel Beam Bridges.* Bulletin No. 29, American Iron and Steel Institute, Washington, D.C., April 1987.
5. *Guide Specifications for Alternate Load Factor Design Specifications for Steel Beam Bridges Using Braced Compact Sections.* American Association of State Highway and Transportation Officials, Washington, D.C., 1986.
6. *Steel Structures - Materials and Design.* Committee Draft TC167/SC CD 10721, International Standards Organization, Geneva, Switzerland, 1991.
7. *Load and Resistance Factor Design Specification for Structural Steel Buildings.* American Institute of Steel Construction, Chicago, Ill. Sept. 1, 1986.
8. G. Haaijer, P. S. Carskaddan, and M. A. Grubb. Autostress Design of Steel Bridges, *Journal of Structural Engineering*, American Society of Civil Engineers, 109, No. 1, Jan. 1983.
9. G. Haaijer. Myths and Realities of Bridge Design. In *Proc., 7th Annual Bridge Conference, Engineers' Society of Western Pennsylvania*, June 18–20, 1990.
10. G. S. Vincent. *Tentative Criteria for Load Factor Design of Steel Highway Bridges.* Bulletin No. 15, American Iron and Steel Institute, March 1969.
11. M. A. Grubb and P. S. Carskaddan. *Autostress Design of Highway Bridges, Phase 3: Initial Moment-Rotation Tests.* Project 188, American Iron and Steel Institute, Washington, D.C., April 18, 1979.
12. M. A. Grubb and P. S. Carskaddan. *Autostress Design of Highway Bridges, Phase 3: Moment-Rotation Requirements.* Project 188, American Iron and Steel Institute, Washington, D.C., July 6, 1981.
13. P. S. Carskaddan. *Autostress Design of Highway Bridges, Phase 3: Interior-Support-Model Test.* Project 188, American Iron and Steel Institute, Washington, D.C., Feb. 11, 1980.
14. C. W. Roeder and Liv Eltvik. *Autostress Design Criteria-Load Test of the Whitechuck River Bridge.* 1985 Final Report, FHWA Project DTFH 61-81-C-00114, University of Washington, Seattle, Jan. 1985.

15. R. P. Knight, M. Moore, and M. A. Grubb. Model Bridge with Precast Concrete Deck Panels. In *Proc., American Institute of Steel Construction National Engineering Conference*, Nashville, Tenn., American Institute of Steel Construction, Chicago, Ill., June 1986.
16. M. Moore and M. A. Grubb. Behavior of a Two-Span Continuous Plate Girder Bridge Designed by the Alternate Load Factor Method. In *Proc., American Institute of Steel Construction National Steel Construction Conference*, Nashville, Tenn., American Institute of Steel Construction, Chicago, Ill., June 1989.
17. C. G. Schilling. Unified Autostress Method. In *Transportation Research Record 1376*, TRB, National Research Council, Washington, D.C., 1992.
18. Agneta Wargsjo and Bernt Johansson. *Plastic Rotation Capacity for Steel Girders with Slender Webs*. Nordic Steel Colloquium, Danish Steel Institute, Copenhagen, Denmark, Sept. 1991.
19. R. P. Johnson and Shiming Chen. *Local Buckling and Moment Redistribution in Class 2 Composite Beams*. Structural Engineering International, International Association for Bridge and Structural Engineering, Vol. 1, No. 4, Nov. 1991.