Autostress Design Using Compact Welded Beams

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INTRODUCTION

After several years of AISI-sponsored research, autostress-design procedures for continuous steel bridges have been incorporated in a 1986 AASHTO *Guide Specification for Alternate Load-Factor Design Procedures for Steel Beam Bridges Using Braced Compact Sections.*[1] Autostress is a procedure that extends existing Load Factor Design (LFD) rules by introducing improved limit-state criteria. LFD is a limit-states design method presently contained in the 13th Edition of the AASHTO *Standard Specifications for Highway Bridges.*[2] The improved limit-state criteria permit inelastic load redistribution in continuous-beam bridges under heavy loads while satisfying the same structural performance requirements as LFD.[3] The autostress procedures in the guide specification are presently limited to rolled-beam bridges (composite and noncomposite), and compact welded-beam bridges that are adequately braced (braced compact sections). AISI is presently sponsoring additional research to extend the autostress procedures to more slender welded plate-girder sections.[4-7]

Autostress-design procedures recognize the ability of continuous steel members to adjust automatically for the effects of local yielding, such as those caused by overloads. Also, the autostress procedures allow a designer to determine the strength of braced continuous compact beams at maximum loads by computing the mechanism resistance using plastic-design theory with some modifications. In both instances, elastic negative bending moments are automatically redistributed by the structure to positive-bending regions. The term autostress has been used for the suggested procedures to emphasize that the load redistribution occurs automatically. In fact, the autostress procedures can simply be viewed as a refinement of the 10 percent redistribution allowance permitted in LFD for compact sections at higher loads. This refinement reduces the gap between present design assumptions and actual continuous-beam behavior. Application of the autostress procedures generally permits a designer to use prismatic steel members in continuous spans along the entire bridge length, or in between field splices. The resulting benefits include lower fabrication costs, and elimination of structural details with undesirable fatigue characteristics. For instance, the need for cover plates can be eliminated from rolled-beam designs. Similarly, flanges of welded beams with similar proportions may have fewer splices and thickness changes.

The development of most of the guide specification provisions was based on test results on compact welded-beam component specimens with proportions similar to rolled beams.[8] The research suggests that structural performance requirements at overload can be satisfied by establishing camber requirements to offset the effects of local yielding in addition to the elastic dead-load deflections. Also, well-established plastic-design principles can be extended to estimate the strength at maximum load by computing the mechanism resistance. Because the mechanism resistance generally is high, permanent deflection (overload) criteria rather than strength (maximum load) criteria usually govern the beam design. This should result in more economical steel-beam bridge designs. The development of the specific guide specification provisions based on the above-mentioned research is discussed in detail in Refs. 8 and 9.

The purpose of this paper is to present the results of comparative preliminary designs of a two-span continuous bridge prepared by AISC Marketing, Inc., for the State of New York. In this study, the bridge was designed using autostress (or alternate load factor), load factor, and working stress (or elastic) design procedures, and rolled shapes and both compact and noncompact welded beams (six schemes total). The study shows that the autostress designs produced the lowest weight steel members and simplest fabrication details for both the rolled-beam and welded-beam solutions. The study therefore suggests that the design solution using a compact welded beam designed by the autostress method, in addition to being the lowest weight design, may indeed be the lowest cost design. To try and confirm this, preliminary cost estimates were obtained for each scheme from two eastern fabricators. These estimates indicate that the autostress-designed welded-beam solution would indeed be the low-cost design. In addition, welded-beam design allows a designer to use a deeper section, which reduces live-load deflections over rolled-beam design. Therefore, compact welded steel beams designed using the guide specification provisions should be given serious consideration in short-span steel bridge design.

GENERAL DESIGN CONSIDERATIONS

Members designed by the LFD method of AASHTO are proportioned for multiples of the design loads. They are required to meet specified structural performance requirements

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at three theoretical load levels—service load, overload, and maximum load.

The left-hand column of Table 1 lists the load levels in order of increasing load. The factor of 1.3 at maximum load is included to compensate for uncertainties in theory, strength, loading, analysis, dimensions, and material properties. The factor of 5/3 is incorporated to allow for overloads.

The second column of Table 1 lists the structural performance requirement for each load level, which is a brief verbal description of the performance required of a bridge at that load level. The guide specification for autostress design requires that members be proportioned to meet the same LFD structural performance requirements at the same three load levels.

Service Load

The guide specification invokes the same limit-state criteria as LFD to satisfy fatigue and live-load performance requirements. Stress ranges are kept within allowable fatigue limits according to the provisions of AASHTO Article 10.3,[2] and elastic live-load deflections may be limited to a specified fraction of the span length (usually $L/800$). Furthermore, the guide specification requires that concrete cracking be controlled by invoking the current AASHTO rules (Article 8.16.8.4) for distribution of flexural reinforcement. Therefore, the design at service load is exactly the same in the load factor and autostress methods.

Overload

At overload, permanent deformations, caused by occasional heavy vehicles, that could be objectionable to the riding quality of the structure must be controlled. In LFD, permanent deformations are controlled by limiting overload flange stresses to $0.80F_y$ in negative bending, and either $0.80F_y$ (noncomposite sections) or $0.95F_y$ (composite sections) in positive bending. F_y is the yield strength of the steel section. For compact sections, 10 percent of the negative interior-pier moment may be redistributed to the maximum positive moment section before computing the stresses.

The autostress procedures in the guide specification place no limit on the stress in negative bending at overload. Instead, elastic overload moments are redistributed to account for inelastic rotation of sections in negative bending at supports. These controlled plastic deformations are allowed to occur in the flange outer fibers at interior supports because they will eventually stabilize after a few cycles of an overload vehicle. The local yielding results in the formation of automoments, so-called because they form automatically due to the continuity of the structure (Figs. 1 and 2). The automoments are residual positive moments that, along with the dead load moments, remain in the structure after the overload vehicle is removed. The automoments reduce the peak elastic interior-pier moments, and ensure that the structure will behave elastically (or shakedown) under all subsequent loads not exceeding the overload. Thus, the stress at the pier sections need not be restricted.

The automoment computations are based on actual moment-rotation behavior of the interior-pier sections. The procedure for determining the amount of inelastic rotation and the magnitude of the corresponding automoment at overload is given in Ref. 8, and is illustrated graphically in Fig. 3. An example computation of the automoment is given in Ref. 1. The procedure is applicable to noncomposite and composite sections. Typical moment-rotation curves are included in the guide specification to aid in the automoment computations if experimental curves are not available (Fig. 4). Permanent deflections caused by the automoments are added to the dead-load deflections in establishing the camber necessary to satisfy the performance requirement. If no overload occurs during the life of the structure, some of the

Table 1. Structural Performance Requirements

Load Level	Structural Performance Requirement
Service Load $[D + (L + I)]$ (nominal dead load plus standard vehicles plus impact)	Provide adequate fatigue life, limit concrete cracking and satisfy live-load deflection requirements.
Overload $[D + (5/3 \times (L + I))]$ (nominal dead load plus occasional heavy vehicles plus impact)	Limit permanent deformations that otherwise could create objectionable riding quality.
Maximum Load $1.3 \times [D + (5/3 \times (L + I))]$ (limited occurrences of factored overload)	Provide load resistance equal to or greater than maximum load.

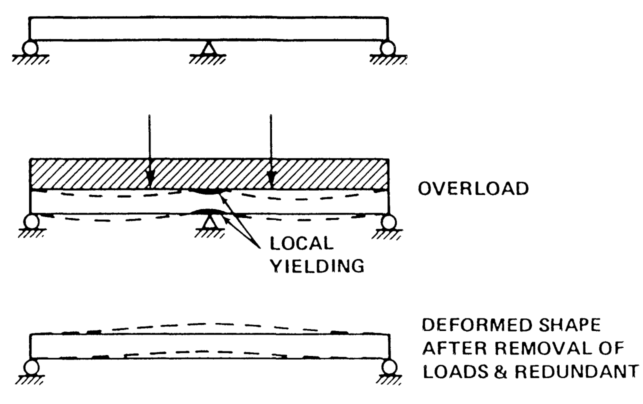

Fig. 1. Deformation caused by overloads.

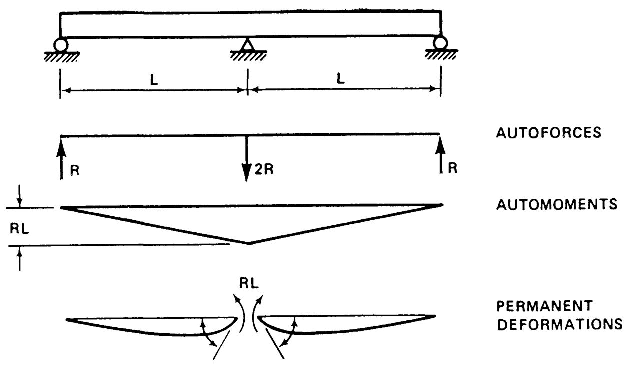

Fig. 2. Automoment diagram and permanent deformations.

automoment camber would remain. However, this would not be objectionable because the automoment camber is relatively small.

In positive bending, the guide specification retains the current LFD overload limit-state criteria to control permanent deformations. That is, the stress on the composite section in positive bending due to the elastic overload moments plus the automoments is limited to $0.95F_y$ for composite sections and $0.80F_y$ for noncomposite sections. The stress due to the automoment should be computed using the section modulus of the composite section, transformed using a modular ratio of $3n$ to allow for creep, since the automoment is considered to be long term. Thus, it can be seen that the automoments are no more than a refinement of the 10 percent redistribution allowed in LFD. They automatically reduce the elastic pier moments and increase proportionately the maximum positive moments.

Maximum Load

The maximum load structural performance requirement is simply that the load must be able to cross the bridge a limited number of times. In LFD, this is achieved for compact sections by limiting the elastic maximum load moment at any section to be below the plastic moment. The plastic moment is equal to the yield strength, F_y, times the plastic section modulus Z (values of Z for rolled sections are given in Ref. 10). Also, for compact sections, 10 percent of the peak negative moments may be redistributed before the comparison is made.

In the guide specification, the maximum load performance requirement is satisfied if the factored load does not exceed the plastic mechanism strength. The calculation of resistance as the plastic-design mechanism strength recognizes the inherent ability of continuous beams with braced compact sections to inelastically redistribute load. In a continuous bridge, the first plastic hinges normally form at the interior piers. To compute the mechanism strength in conventional plastic design, the pier sections must be able to reach the plastic moment, and the hinges at the piers must be able to rotate inelastically at the plastic moment as the load redistributes to the positive moment sections. To accomplish this, the flange- and web-slenderness ratios must be within specified limits, and the lateral bracing of the compression flange must be adequate.

In the guide specification, a compact section is defined as a section that can both reach the plastic moment and rotate inelastically at the plastic moment a limited amount. For a steel section to qualify as a braced compact section so that the maximum strength can be determined from a plastic mechanism analysis, the guide specification requires that the section meet the following criteria: 1) the slenderness ratio of the projecting compression-flange element must meet the limit

$$\frac{b'}{t} \leq \frac{2055}{\sqrt{F_{yf}}} \tag{1}$$

where b' is the width of the projecting flange element, t is the flange thickness, and F_{yf} is the yield strength of the

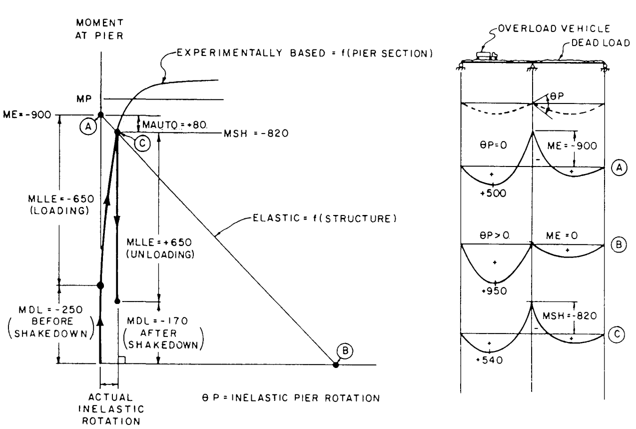

Fig. 3. Graphic determination of automoment and permanent rotation.

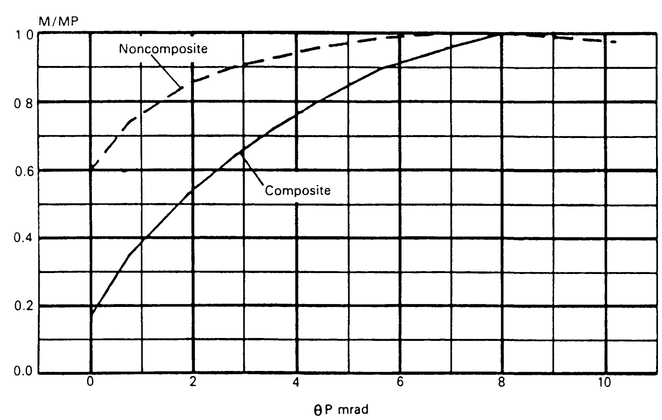

Fig. 4. Guide specification moment vs. permanent rotation curves—overload.

compression flange in psi, and 2) the web slenderness must meet

$$\frac{D}{t_w} \leq \frac{19{,}230}{\sqrt{F_{yf}}} \tag{2}$$

where D is the distance between flanges and t_w is the web thickness. F_{yf} is used for the web because plastic web buckling is governed by flange strain. In Eq. 2, D should be replaced by $2D_{cp}$ for unsymmetrical sections in negative bending where the distance from the neutral axis to the compression flange exceeds $D/2$. D_{cp} is the distance to the compression flange from the neutral axis at the plastic moment. The above flange and web limits are 9.2 and 86.0, respectively, for 50,000 psi steel (the maximum yield-strength steel for which the guide specification provisions can presently be applied). When the flange and web slenderness ratios both exceed 75 percent of the limits in Eqs. 1 and 2, the following interaction equation[11] is given in the guide specification to redefine the allowable limits:

$$\frac{D}{t_w} + 9.35\left(\frac{b'}{t}\right) \leq \frac{33{,}650}{\sqrt{F_{yf}}} \tag{3}$$

Once again, D should be replaced by $2D_{cp}$ if the distance from the neutral axis to the compression flange exceeds $D/2$.

Lateral bracing of the compression flange adjacent to a rotating hinge is also an important consideration. The bracing helps to prevent lateral buckling of the flange, which has a detrimental effect on rotation capacity. The lateral bracing requirements in the guide specification are adopted directly from the AISC LRFD Specification.[10] The limit for the required lateral bracing is given as:

$$\frac{L_b}{r_y} \leq \frac{[3.6 - 2.2\ (M_1/M_2)] \times 10^6}{F_y} \tag{4}$$

L_b is the distance between points of bracing of the compressing flange; r_y is the radius of gyration of the steel beam with respect to the Y-Y axis; F_y is the yield strength in psi, and M_1 and M_2 are the moments at the two adjacent brace points. M_1/M_2 is positive for a member bent in single curvature. A trial-and-error procedure is required to determine L_b. For compact sections, the guide specification also requires that bearing stiffeners be located at rotating hinge locations.

Braced compact sections with flange- and web-slenderness ratios in the upper range of the above slenderness limits are able to reach the plastic moment, but may not have enough available inelastic rotation capacity at the plastic moment. Rather than limit the number of sections that can be used, the guide specification requires that an effective plastic moment[12] M_{pe} be computed for the interior pier sections to account for the effects of local web and flange buckling. M_{pe} is a reduced moment, based on the actual section geometry, at which the pier sections can be considered to have adequate rotation capacity. For compact sections with flange- and web-slenderness ratios in the lower range of the specified slenderness limits, M_{pe} will equal the plastic moment. M_{pe} is simply used in place of the full plastic moment at the piers in the mechanism analysis. Adequate rotation capacity is ensured, and therefore, does not have to be computed. Since the interior-pier sections are assumed to rotate at a constant moment equal to M_{pe}, the maximum moments in positive bending can be easily computed from the statics of simple beams with end moments equal to M_{pe}. If the computed maximum positive moments are less than the respective maximum strengths in positive bending, the design is satisfactory at maximum load. The computation of the effective plastic moment is illustrated in Refs. 1 and 12. A sample mechanism calculation is given in Ref. 1.

For compact sections in positive bending, the maximum strength at maximum load according to the guide specification is computed assuming a fully plastic stress distribution in the sections, including the concrete slab. The moment capacity then equals the sum of the moments about the plastic neutral axis of the composite section of all compressive and tensile forces. The guide specification also states that whenever a composite section in positive bending reaches this plastic moment, no further plastic rotation is permitted. Thus, if the first hinge should form in positive bending, the limit state is reached.

COMPARATIVE PRELIMINARY DESIGNS

A comparative preliminary design study was completed by AISC Marketing, Inc., for a two-span continuous structure in Chautauqua County, New York. The study presented comparisons of six different schemes using rolled shapes and both compact and noncompact welded beams designed using autostress (guide specification), load factor, and working stress procedures. The six schemes were as follows:

- SCHEME A: Compact welded beams designed using autostress procedures.
- SCHEME B: Compact rolled beams designed using autostress procedures.
- SCHEME C: Noncompact welded plate girders designed using load factor procedures.
- SCHEME D: Compact rolled beams designed using load factor procedures.
- SCHEME E: Welded plate girders designed using working stress procedures.
- SCHEME F: Rolled beams designed using working stress procedures.

The two-span structure had a total length of 164′–0″ and individual span lengths of 82′–0″. The bridge cross section was composed of four beam lines spaced at 11′–0″ on centers, supporting a 9.75-inch concrete deck slab which cantilevers 4′–6″ beyond the exterior beams (Fig. 5). The use of an 11′–0″ beam spacing allowed for the elimination of one beam line, which added further to the economy of the structure. Stay-in-place forms for the concrete deck were used to span between the beams.

In determining elastic moments and shears, the concrete slab was considered to be effective in distributing moments in the positive-moment region only. For determining section properties for resisting moments and shears, the structural concrete slab was considered effective only in the positive-bending regions. The steel section alone was considered effective in negative-bending regions. Additional economy could have been achieved by adding stud shear connectors in the negative-moment region so that composite action between the steel beam and rebar could be assumed.

The bridge was designed for AASHTO HS25 live loading. The elastic live-load moments and shears were modified by the appropriate AASHTO live-load distribution factors. Studies have shown that additional economies could also have been achieved by determining more rational lateral-distribution factors using mathematical three-dimensional finite-element models.[13, 14] For fatigue, a Case III roadway was specified, which meant the bridge was to be designed for 100,000 cycles of HS25 truck and lane loading. A live-load deflection limit of $L/800$ was specified for the bridge. Unpainted A588 structural steel was used for all beams, stiffeners, and diaphragms or cross frames. Superstructure concrete with a specified 3000 psi 28-day compressive strength was used with 60 ksi reinforcing bar. Reinforcing bar with a yield strength of 60 ksi is recommended in the autostress procedures so that the rebar will remain elastic at overload.

Scheme A

Scheme A represents a solution utilizing compact welded beams designed using the guide specification provisions. A beam-line elevation of an interior beam is shown in Fig. 6. This scheme shows how a prismatic welded steel section with no transverse web stiffeners can be used. Cross frames were spaced adjacent to the interior pier according to the guide specification requirement given by Eq. 4, discussed above. It is possible that additional economy may have been achieved by bending plates into channel shapes with a depth approximately one-third to one-half the beam depth to replace the labor-intensive cross frames; this would require further study. Bearing stiffeners were required at the abutments and interior pier. The beam is cambered to include the anticipated deflections due to the automoment at overload. These deflections are computed for the elastic deflection formula for a simple-span beam with an end moment equal to the automoment.

A web plate 48 in. deep and $\frac{9}{16}$ in. thick was used throughout. Optimization of the web depth would require the preparation and comparison of several designs, which was not practical without appropriate computer software for autostress design (not available at this writing). Therefore, a 48-in. web depth was selected based on examination of various load factor designs for a similar cross-section configuration and similar span lengths. It should be emphasized that this was only an estimation, and does not imply that the optimum web depth is the same for autostress and load factor design; further study is required.

The 12 in. by $\frac{5}{8}$ in. top flange plate and 12 in. by $\frac{7}{8}$ in. bottom-flange plate used throughout were selected with the aid of established rules-of-thumb for steel-beam fabrication.[15] The minimum flange width of 12 in. was established based on erection considerations for the shipping pieces. The minimum flange thickness of $\frac{5}{8}$ in. was selected to minimize flange distortion during welding of the flange to the web plate. Trial flange plates were then tried until the beam satisfied the overload limit state.

Complete detailed design calculations for this scheme are not presented here, but are similar to those presented in the design example furnished in Ref. 1. The maximum stress in positive bending at overload (including the stress due to the automoment) was computed to be $0.92F_y$, which is below the guide-specification limit state. The fact that the overload

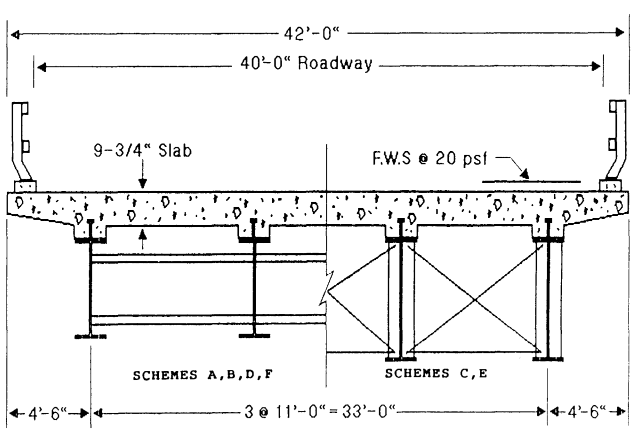

Fig. 5. Cross section.

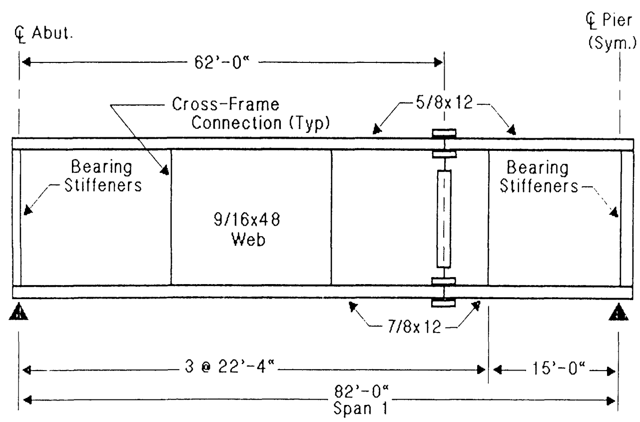

Fig. 6. Scheme A: Alternate load factor (autostress) design—compact welded beam.

criterion governs the design is typical of most designs done according to guide specification rules, which makes the overload check a good place to start any preliminary autostress design. The mechanism analysis showed significant reserve strength at maximum load. The effective plastic moment for the pier section used in the mechanism analysis was computed to be 80 percent of the plastic moment of the steel beam. The live-load deflection at service load (HS25 loading) was computed to be $L/1080$. The compact welded beam allowed for the use of a deeper section than a rolled beam, which significantly reduced the live-load deflection over the rolled-beam designs to be discussed in the following sections (Schemes B, D, and F). Fatigue did not govern. The total weight of the structural steel in Scheme A, including cross frames and connection plates, bearing stiffeners, and field splices, was approximately 55.9 tons. This is equivalent to a low unit weight of 16.1 psf on deck area. Also, minimal fabrication would be required.

Scheme B

Scheme B represents a solution utilizing compact rolled beams designed using the guide specification provisions. A beam-line elevation of an interior beam is shown in Fig. 7. This scheme shows how a continuous W36×210 rolled shape without cover plates can be used. The diaphragms are rolled channels that were spaced the same as in Scheme A. Deeper bent-plate channels could again be considered for greater economy and stability. Note that bearing stiffeners were required at the interior-pier section according to the guide specification. The beam camber will again include the anticipated deflections due to the computed automoment at overload.

The design calculations were again similar to those in Ref. 1. The calculations showed that the overload limit-state criterion governed the design. The maximum stress in positive bending at overload (including the stress due to the automoment) was equal to $0.92F_y$, which is less than the specified guide-specification limit state of $0.95F_y$. The mechanism analysis at maximum load showed that the bridge has significant excess strength. The live-load deflection at service load (HS25 loading) was right at the $L/800$ limit, which is approximately 26 percent greater than the live-load deflection in Scheme A. Again, fatigue did not govern. The total weight of the structural steel in Scheme B, including the diaphragms and connection plates, bearing stiffeners, and field splices, was approximately 73.4 tons. This is equivalent to a unit weight of 21.1 psf of deck area.

Scheme C

Scheme C represents an optimized design solution utilizing noncompact welded plate girders designed using the load factor method. A girder-line elevation of an interior girder is shown in Fig. 8. This scheme illustrates the use of a nonprismatic welded steel plate girder with transverse stiffeners on one side of the web. Flange transitions (circled in Fig. 8) were located to maximize the performance of the girder measured against the specified limit states. At each transition, it was ensured that there was no more than a 50 percent reduction in flange area across the transition. Cross frames were equally spaced at 20′-6.″ The girder was therefore designed as an unbraced noncompact girder according to the provisions of AASHTO Article 10.48.4. Bearing stiffeners were required at the interior pier and abutments.

The girder was designed using the steel-girder-design computer program, SIMON, that was developed by the USX Corporation.[16] The girder depth was optimized using a pre- and post-processor to SIMON. A 45-in. web depth was selected. The web thickness was varied from $\frac{7}{16}$ in. in each span to $\frac{1}{2}$ in. in the field piece over the interior pier. Three different flange thicknesses were required for the top and bottom flanges in each span. The flange width of the top and bottom flange in each section was kept the same for ease of fabrication. The minimum flange width and thickness was

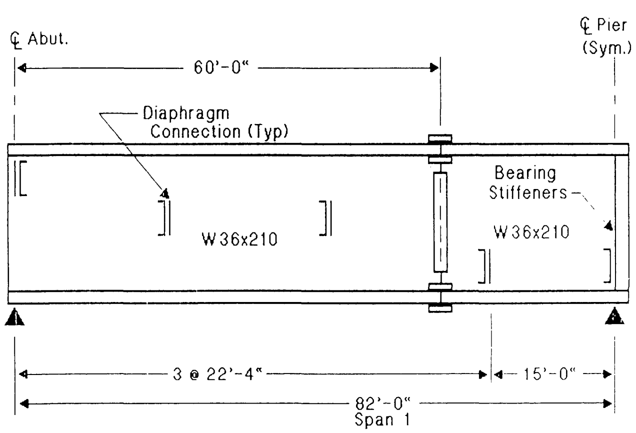

Fig. 7. Scheme B: Alternate load factor (autostress) design—rolled beam.

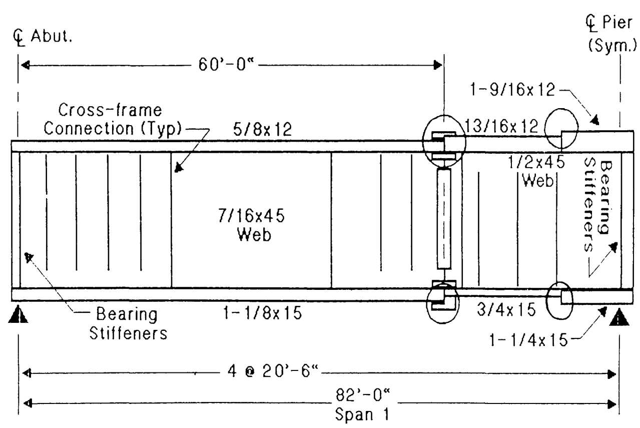

Fig. 8. Scheme C: Load factor design—non-compact plate girder.

selected based on the same criteria used in Scheme A.

The design was governed by the bottom-flange bending stresses at maximum load. The bottom-flange bending stresses at the maximum positive moment section and over the interior pier were both approximately equal to the yield stress, which is the limit state. It can be demonstrated that overload criteria never govern for noncompact plate girders designed by the load factor method. The live-load deflection at service load (HS25 loading) was computed to be *L*/1144. The total weight of the structural steel in Scheme C, including cross frames and connection plates, bearing stiffeners, and field splices, was approximately 56.7 tons. This is equivalent to a unit weight of 16.3 psf of deck area. However, additional fabrication is required over the welded-beam design in Scheme A because there are more flange transitions, web splices, and transverse web stiffeners.

Scheme D

Scheme D represents a solution utilizing compact rolled beams designed using the load factor method. A beam-line elevation of an interior beam is shown in Fig. 9. This scheme uses a W36×210 rolled shape in each span, and a W36×245 rolled shape (without cover plates) in the field piece over the interior pier. The diaphragms are rolled channels. They were spaced adjacent to the interior pier according to criteria in AASHTO Article 10.48.1.1(c) so the W36×245 would qualify as a compact section. Since both sections qualified as compact, the 10 percent redistribution rule was applied to the elastic moments before checking moments and stresses against the limit states. Bearing stiffeners were not required.

The beam was designed using the steel-beam-design computer program, PIPER, that was developed for the USX Corporation.[17] The design was governed by the bending stresses in the flanges at overload (after moment redistribution). The top- and bottom-flange bending stresses at the interior pier were approximatley equal to $0.80F_y$, which is the overload limit state in load factor design for the noncomposite steel section at the pier. The bottom-flange bending stress at the maximum positive moment section was approximately equal to $0.91F_y$, which is less than the overload limit state of $0.95F_y$ for a composite section. The applied elastic maximum load pier moment after redistribution was approximately 93 percent of the plastic moment of the steel beam, and the elastic maximum positive moment at maximum load after redistribution was approximately 85 percent of the plastic moment of the composite section. This satisfies the maximum load criterion in load factor design. The live-load deflection at service load (HS25 loading) was computed to be *L*/867. The total weight of the structural steel in Scheme D, including diaphragms and connection plates, and field splices, was approximately 76.2 tons. This is equivalent to a unit weight of 21.9 psf of deck area.

Scheme E

Scheme E represents an optimized design solution utilizing welded plate girders designed using working stress or traditional elastic-design procedures. A girder-line elevation of an interior girder is shown in Fig. 10. A nonprismatic welded steel plate girder with transverse stiffeners on one side of the web was required. The girder was designed using the steel-girder-design program SIMON, discussed previously. The design is essentially the same as the welded-girder design presented in Scheme C (load factor design), except that thicker flange plates were required in negative-moment sections in the working stress design. The weight differential between the load factor and working stress designs (Schemes C and E) would probably be larger for longer spans. The live-load deflection at service load (HS25 loading) was computed to be *L*/1215. The total weight of the structural steel in Scheme E, including cross frames and connection plates, bearing stiffeners, and field splices, was approximately 58.8 tons, equivalent to a unit weight of 16.9 psf on deck area.

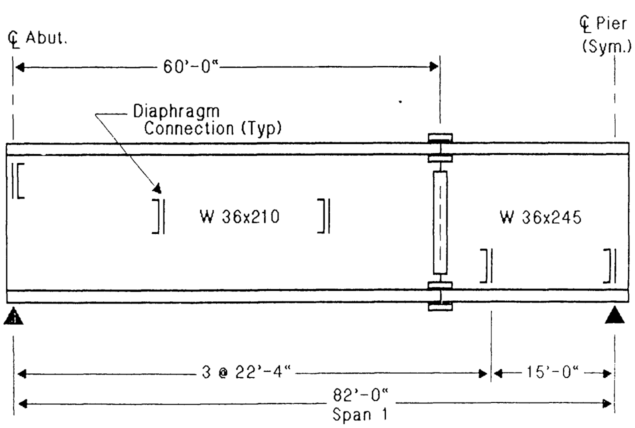

Fig. 9. Scheme D: Load factor design—rolled beam.

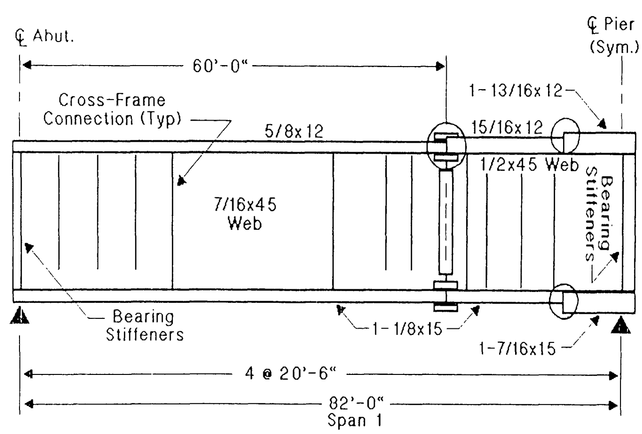

Fig. 10. Scheme E: Working stress design—plate girder.

Scheme F

Scheme F represents a solution utilizing rolled beams designed using working stress or traditional elastic-design procedures. A beam-line elevation of an interior beam is shown in Fig. 11. This scheme uses a W36×230 rolled shape in each span, and a W36×328 rolled shape (without cover plates) in each field piece over the interior pier. Again, the diaphragms are rolled channels. The channels are equally spaced at 20′-6″ since the 10 percent redistribution rule is not applicable in working stress design. Bearing stiffeners were not required. The live-load deflection at service load (HS25 loading) was computed to be *L*/965. The total weight of the structural steel in Scheme F, including diaphragms and connection plates, and field splices, was approximatley 88.2 tons. This is equivalent to a unit weight of 25.4 psf of deck area. Scheme F was the heaviest of the six schemes.

RESULTS AND CONCLUSIONS

The improved limit-state criteria presented in the AASHTO guide specification for braced compact sections should result in demonstrated economies for short-span steel bridge design. In this study, six comparative preliminary designs were prepared for a two-span continuous structure (82′-82′) using rolled shapes and both compact and noncompact welded beams designed by autostress (alternate load factor), load factor, and working stress (elastic) procedures.

The study showed that for both rolled-beam and welded-beam solutions, the autostress designs produced the lowest weight steel members and simplest fabrication details. As shown in Table 2, the non-optimized compact welded-beam solution designed by the guide specification provisions (Scheme A) had the lowest unit weight of structural steel at an efficient 16.1 psf. The optimized noncompact plate-girder solution designed using load factor procedures (Scheme C) was only 1.2 percent greater in weight at 16.3 psf. The next heavier solution was the plate-girder solution designed using working stress procedures (Scheme E), which was 5.0 percent greater in weight at 16.9 psf. The rolled-beam solution designed using the guide specification provisions (Scheme B) was 31.1 percent greater in weight at 21.1 psf. The rolled-beam solution designed using load factor procedures (Scheme D) was 36.0 percent greater in weight at 21.9 psf. Finally, the rolled-beam solution designed using working stress procedures (Scheme F) was 57.8 percent greater in weight at 25.4 psf. All weights include the estimated weight of all bracing, stiffeners, and field splices.

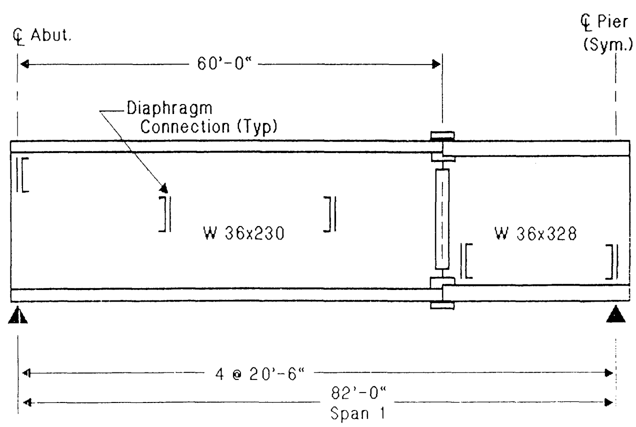

Fig. 11. Scheme F: Working stress design—rolled beam.

The demonstrated difference in unit weights may not reflect the difference in costs between the six design solutions. The autostress procedures do not necessarily always yield large weight savings, or even the minimum-weight design over other design methods. Instead, the procedures aim to maintain the simplicity of the completed steel structure to minimize total cost including fabrication. For example, Scheme A eliminates labor-intensive fabrication costs associated with 24 flange splices, 4 web shop splices, and 72 transverse web stiffeners that exist in Schemes C and E, even though the weight differential between Schemes A and C is only 1.2 percent and the weight differential between Schemes A and E is only 5.0 percent. Scheme B eliminates four shop splices between rolled shapes, in addition to being 3.7 percent lighter than Scheme D and 20.4 percent lighter than Scheme F. The weights of Schemes D and F could have been reduced by utilizing cover plates; however, this advantage would be offset somewhat by increased fabrication costs. The autostress design solutions also provide for better performing structures less susceptible to fatigue damage. Therefore, the autostress design solutions would appear to be the most cost-effective rolled-beam and welded-beam solutions. The relative economy of the autostress rolled beam and welded beam solutions depends largely on the price differential between rolled shapes and plate. However, the compact welded beam solution is 23.7 percent lighter than the rolled beam solution.

To obtain an indication of the relative costs of the six schemes, preliminary cost estimates for all schemes were obtained from two eastern fabricators. Table 2 indicates the relative costs of each scheme in terms of an average normalized cost index. This index represents the average of the cost estimate from each fabricator for the scheme divided by the average cost estimate for the low-cost scheme. All costs are for the fabricated steel erected in place. The design by autostress procedures (Scheme A) is 8 percent less costly than the optimized noncompact plate-girder solution designed using the load factor method (Scheme C), and 12 percent less costly than the optimized plate-girder solution designed using the working stress method (Scheme E). It is conceivable that this cost differential could have been increased had it been possible to optimize the autostress welded-beam design. The autostress rolled-beam solution (Scheme B) is 15 percent more costly than Scheme A followed by the load factor rolled-beam solution (Scheme D), which is 23 percent more costly than Scheme A. The working stress rolled-beam solution (Scheme F) is 42 percent more costly than Scheme A. It should also be noted that Scheme B is 6.5 percent less costly than Scheme D and 19 percent less costly than Scheme F, indicating that the autostress rolled-beam solution is the most cost-effective rolled-beam design. It is recognized that regional differences could influence these relative costs.

In summary, the design of short-span steel bridges using the autostress procedures contained in the guide specification should provide demonstrated economies and better performing structures over designs prepared by the load factor and working stress methods. Welded-beam design should be considered because it allows a designer to use a deeper section, which reduces live-load deflections over rolled-beam design. The low unit weight and ease of fabrication of these welded beams may also provide a greater competitive edge over rolled beams against precast, prestressed concrete for short-span bridges.

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Table 2.
Preliminary Design Weight and Cost Comparisons

	Schemes					
	A	B	C	D	E	F
Unit Weight of Structural Steel—psf	16.1	21.1	16.3	21.9	16.9	25.4
Total Weight (incl. bracing, etc.)—tons	55.9	73.4	56.7	76.2	58.8	88.2
Normalized Weight Index	1.00	1.31	1.01	1.36	1.05	1.58
Average Normalized Cost Index	1.00	1.15	1.08	1.23	1.12	1.42

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