BRIDGE DESIGN BY THE AUTOSTRESS METHOD

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The Autostress method has been evolving as an extension to the AASHTO Load-Factor method for rolled-beam and plate-girder steel highway bridges. The Autostress method uses the same three load levels as the Load-Factor method: Service Load, Overload, and Maximum Load. However, to satisfy the structural performance requirements, the Autostress method injects two new concepts into bridge design: mechanism formation at Maximum Load and shakedown at Overload. When a structure forms a mechanism, there are sufficient plastic hinges to cause failure. When a structure shakes down, residual stresses and residual moments are automatically developed and assure elastic behavior under subsequent loading—hence the term Autostress. Although results of both methods for a simple span are the same, the Autostress method provides economies in continuous-span bridges; it utilizes the same safety factor against mechanism formation in both simple-span and continuous-span bridges. In contrast, the Load-Factor method uses a higher safety factor in continuous-span bridges than in simple-span bridges. As part of an AISI-sponsored project, nine Load-Factor bridges were redesigned according to the Autostress method; the average cost saving was 10.7 percent. The objective of the AISI project is to suggest that the Autostress method be incorporated into the AASHTO specification after experimental verification.

During the initial few passages of a sufficiently heavy load, a structure designed according to the Autostress-Design method will develop autostresses and automoments caused by local plastic deformations. During subsequent passages of a similar load, these automoments will assure elastic behavior. Automoments are analogous to the moments induced in a concrete beam by prestressing operations; the main difference from concrete prestressing is that a continuous steel beam requires no tendons and develops the required automoments automatically—hence the name Autostress Design.

The Autostress-Design (ASD) method was initially conceived(1) to utilize the ability of a redundant steel structure to shake down—that is, to develop automoments, which permit subsequent elastic behavior. In applying the shakedown principle to bridge design, a related principle—mechanism formation—was also introduced into the ASD method. Today, the ASD method is an extension of the Load-Factor-Design (LFD) method. The American Iron and Steel Institute (AISI), believing that ASD could be fruitfully applied to highway-bridge design, sponsored Project 188, "Autostress Design of Highway Bridges." This paper will review the progress of AISI Project 188: the loadings, the limit-state criteria, some sample designs, and the proposed future work. These topics are more completely discussed in two available reports.(2,3)

Loadings and Limit States

Loadings

The current AASHTO specification(4) permits use of the LFD method. The LFD method approximates the actual behavior of bridges more closely than the Working-Stress-Design method that was previously used for all bridges. As the name suggests, LFD applies factors to the loadings; the two main bridge-load types are dead load, D, and service live load plus impact, L+I. By applying factors individually to dead and live loads, a more uniform safety margin for live loads is achieved among different span lengths because the ratio of D/(L+I) varies with span length.

The LFD method specifies three levels of loading—Service Load, Overload, and Maximum Load—and each load level is associated with appropriate load factors. The second column of Table 1 lists these three load levels in order of increasing load. The third column of Table 1 lists for each load level a structural performance requirement, which is a brief verbal description of the performance required of a bridge at that load level.

Standard vehicles plus nominal dead load are applied at Service Load; for this condition a bridge should have adequate fatigue life, controlled elastic response to live loads, and limited concrete-deck cracking. Occasional permit vehicles, specified by AASHTO as having 5/3 times the weight of a Service-Load vehicle weight, plus nominal dead load are applied at Overload; a bridge should have a riding quality not rendered objectionable due to permanent deformations caused by yielding. A few emergency passages of exceptionally heavy vehicles, specified by AASHTO as having 1.3 times the weight of an Overload vehicle, plus a 30 percent increase in dead load are applied at Maximum Load; a bridge must support these loads although significant permanent damage may result.

The ASD method is applicable to the same three load levels as those specified by AASHTO in the LFD method. However, the ASD method may also be used for different load-level magnitudes that more closely represent the loadings of a particular situation, such as on a restricted route subjected to heavier-than-normal loadings.

Limit States

A limit-state criterion is a constraint that assures a desired structural performance. In Table 1 are listed, for each associated load level, the LFD and ASD limit-state criteria. The criteria for both methods are intended to satisfy the same structural performance requirements; these criteria are discussed and compared below for each load level. Although only certain criteria will govern a design, all criteria must be checked because the governing criteria may not be predictable. Thus, an acceptable design must satisfy all the limit-state criteria associated with the design method.

Service Load. The same limit-state criteria are used for LFD and ASD to satisfy fatigue and live-load response requirements; for fatigue the stress ranges cannot exceed values based on loading cycles and type of detail, and for live-load response the elastic live-load deflection is usually limited to some fraction of the span length. Automoments do not affect live-load stress ranges or elastic deflections. The structural-performance requirement for limited concrete-deck cracking is shown parenthetically in Table 1 to indicate that such a requirement is not explicitly required in the LFD method. However, as will be discussed under Overload, a control on maximum crack width is desired in the ASD method. Thus, AASHTO Equation 6-30, which is under the concrete LFD portion of the specification,(4) is invoked. This equation ensures a limit on the maximum crack width by setting the allowable rebar stress as a function of the rebar distribution; generally, many small bars permit a higher allowable rebar stress than do a few large bars.

Overload. The Overload structural-performance requirement—control of permanent deformations—deals only with serviceability; safety is not considered. The LFD method controls permanent deformations by limiting a flange stress due to negative moment to 0.80FY, where FY is the flange yield strength, and by limiting a flange stress due to positive moment to 0.95FY for composite sections and 0.80FY for noncomposite sections. If certain compactness requirements are met in a continuous-span bridge, the LFD method permits ten percent of the negative pier moment to be redistributed prior to making the stress calculations; the actual moments are generally below the yield moment.

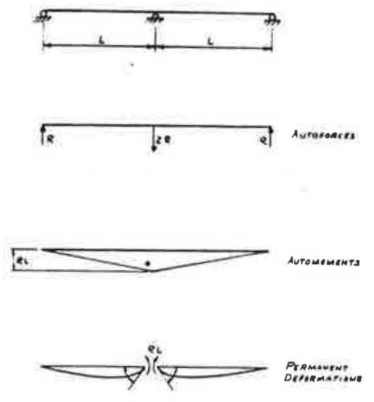
The ASD method controls permanent deformations by permitting a continuous-span bridge to shake down—that is, to undergo small plastic deformations at a pier that will stabilize after a few cycles. These plastic deformations normally occur only in the flange outer fibers—they do not create a plastic hinge. After a bridge has shaken down, it will respond elastically to all subsequent loads not exceeding the Overload. In contrast, a structure that is repeatedly loaded above the shakedown load will fail by incremental collapse: ever increasing deflections caused by increments of plastic deformation in the same direction.(5) However, the greatest load at which a bridge will shake down—the shakedown load—would create deformations objectionable to riding quality. A bridge will shake down at any load less than the shakedown load, and the permanent deformation increases as the load approaches the shakedown load. Thus, the ASD method imposes additional limit states that keep the maximum Overload below the shakedown load. As a result, the interior-pier negative moment is usually above the yield moment but below the plastic moment—although this is not a limit-state criterion.

In the process of shaking down, a continuous structure develops a set of residual forces and moments, as shown in Figure 1, which are in equilibrium when the structure is unloaded. These moments, termed automoments because they are automatically developed, ensure that the structure remains elastic when subjected to Overload and may be considered as a modification of the dead-load moments; for a two-span bridge the automoment reduces the pier dead-load moment. Automoments are determined as the difference between the yielded moment distribution and the elastic unloading Overload moment.(3) The AASHTO Specification provisions for hybrid beams implicitly recognize the beneficial effect of autostresses in assuring elastic behavior after initial local plastic deformations caused by

Table 1. Comparison of Load Factor Design and Autostress Design
(Rolled-beam and plate-girder bridges supporting HS20 or H20 loading)

Load Factor Design Limit-State Criteria (Compact Section)	Load Level	Structural Performance Requirement	Autostress Design Limit-State Criteria
Stress ranges shall be less than categorized limits. Live-load deflection shall be less than a rational amount.	Service Load [D + (L+I)] (standard vehicles)	Provide adequate fatigue life, control elastic live-load deflections, (and limit concrete cracking).	Stress ranges shall be less than categorized limits. Live-load deflection shall be less than a rational amount. Rebar allowable stress shall be based on the rebar distribution within the slab.
Stresses due to positive and negative moments shall be less than a percentage of the yield stress; a 10% redistribution of elastic moments in continuous spans is permitted.	Overload [D + 5/3(L+I)] (occasional permit vehicles)	Control permanent deformations that otherwise could create objectionable riding quality.	A bridge shall shake down, that is, reach a condition in which further loadings cause no further yielding. Automoments may develop. Inelastic rotation at an interior support due to an automoment shall be less than a certain angle. Stresses due to positive moment—applied moment plus automoment—shall be less than a percentage of the yield stress. Rebar yielding at interior supports shall be minimized by using 414 MPa (60 ksi) rebars and unshored construction.
The moment at any cross section shall not exceed the plastic moment capacity; a 10% redistribution of elastic moments in continuous spans is permitted. Criteria related to the following parameters shall be used to assure that the plastic moment can be reached. a. compression-flange width/thickness b. compression-flange slenderness c. web depth/thickness d. shearing force	Maximum Load 1.3[D + 5/3(L+I)] (few emergency passages of exceptionally heavy vehicles plus additional dead load)	Resist loads that may cause significant permanent damage.	A mechanism shall not form; consider uplift. At plastic hinges, except the last to form, the following shall be limited: a. compression-flange width/thickness b. compression-flange slenderness c. web depth/thickness d. shearing force e. cross-section distortion (add a stiffener) Away from plastic hinges, the members shall be able to transmit the moments and shears. No additional load shall be permitted when a hinge forms in positive bending.

Figure 1. Typical automoment diagram for a two-span bridge.



local web yielding. Although the ASD method may consider automoments to meet the Overload performance requirements, the bridge may never experience automoments unless the loading is sufficiently heavy; such behavior is acceptable.

To control permanent deformation at a pier due to the automoment, inelastic rotation, shown in Figure 1, shall not exceed the tolerance to which the slab is constructed: about 3 mm (1/8 in.) in 3 m (10 ft). The angle caused by inelastic rotation is easily computed; for a two-span bridge, as shown in Figure 1, the inelastic-rotation angle equals the end rotation of a simple beam with the indicated end moment of RL. If this were a composite bridge, the composite steel-concrete moment of inertia should be used since the automoment is positive along the entire span. The deflection in the span due to this inelastic rotation is small and may be included in the dead-load camber.

Permanent deformation caused by yielding in regions of positive bending is not directly controlled. Instead, stresses are controlled by the same limit state as that used in the LFD method for two reasons: one is that a better criterion is not known because small amounts of yielding in positive bending may create significant deformations; the other is that the same Overload limit state governs for both methods in a simple-span bridge.

During the development of an automoment, maximum crack width in the concrete deck is better controlled if the rebars remain elastic. This may be achieved if 414 MPa (60 ksi) yield-strength rebars and unshored construction are used. In unshored construction, the dead-load stresses in the steel section are higher than those in the rebars because the steel section is subjected to the full dead load and the rebars are subjected to only the composite dead load. Thus, any yielding necessary to create the automoment will desirably occur entirely within the steel section and the rebars will remain elastic. If this occurs, the rebar dead-load tensile stress will be reduced by the automoment.

Maximum Load. The Maximum-Load structural performance requirement—load resistance—deals only with safety; serviceability is not considered. The LFD method requires the Maximum-Load moment at any section in a bridge to be below either the yield or the plastic moment. These moments are elastic moments with the same possible ten-percent redistribution as mentioned above for Overload. In the LFD method, a simple-span bridge with a compact section reaches its maximum carrying capacity when a plastic hinge forms near midspan. This hinge, together with the two true hinges at the supports, forms a mechanism and prevents additional loading. Plastic design is therefore permitted in LFD for simple spans.

The plastic-design concept of mechanism formation permitted by the LFD method for simple-span bridges is extended to continuous-span bridges in the ASD method. In a two-span continuous beam a plastic hinge usually develops first at an interior pier, but this hinge is not sufficient to form a mechanism. Additional loading is required to cause a second plastic hinge at about midspan. The two plastic hinges, together with the true hinge at the exterior support, form a mechanism and prevent additional loading. During the formation of the second hinge, the interior-support plastic hinge must rotate inelastically at the plastic moment. To ensure that this inelastic rotation can occur while maintaining the plastic moment, the special limit-state criteria indicated in Table 1 are required. In general, these criteria are necessary at all plastic hinges except the last to form; at the last plastic hinge no inelastic rotation is required because the mechanism-formation limit state has been reached. The AISC specification(6) contains such plastic-hinge criteria, but since the amount of inelastic rotation required in a highway bridge is less than that required in a building, new, less restrictive plastic-hinge criteria are being developed for bridges.

A designer must ensure that uplift of an end reaction does not occur during mechanism formation. A downward reaction may be required to satisfy equilibrium and to prevent the ends of the bridge from lifting off their supports. Field testing of a bridge showed that uplift may occur prior to ultimate load.(7)

Away from the plastic hinges, the moments and shears from Maximum Load must not exceed those permitted by the LFD method. Since the inelastic-rotation capacity of a composite section is not accurately known (because of possible concrete crushing),(8) an additional limit-state criterion is imposed: additional load shall not be permitted when a plastic hinge has formed in positive bending. This criterion will not govern the usual bridge.

Design Study

Ten bridges designed by the LFD method were redesigned according to the ASD method to determine the relative weights and costs. The study(2) included rolled beams of constant and variable sections, a cover-plated rolled beam, plate girders, and composite and noncomposite bridges. Relative to the LFD method, the ASD method produced an average weight saving of 12.4 percent and an average cost saving of 10.7 percent. The relative cost savings are plotted versus span length in Figure 2. All bridges were two-span continuous designs except for Case D, which was a simple-span design, and Case F, which was a four-span continuous design. The three designs discussed below illustrate how a LFD bridge compares with an ASD bridge.

Case A

Figure 3 shows both the LFD and ASD bridges. The LFD bridge is a constant-section rolled beam with top and bottom cover plates at the pier. The ASD bridge uses the same section but the cover plates are eliminated. For comparison, the LFD bridge at the pier was designed as an unbraced section. The ten-percent moment redistribution could not therefore be used and the beam size plus cover plates was governed by the Maximum-Load limit state. The ASD bridge at the pier is governed by the limit states of the inelastic rotation at Overload; the rebar stress range for fatigue at Service Load; and the

Figure 2. Autostress-design cost reductions.

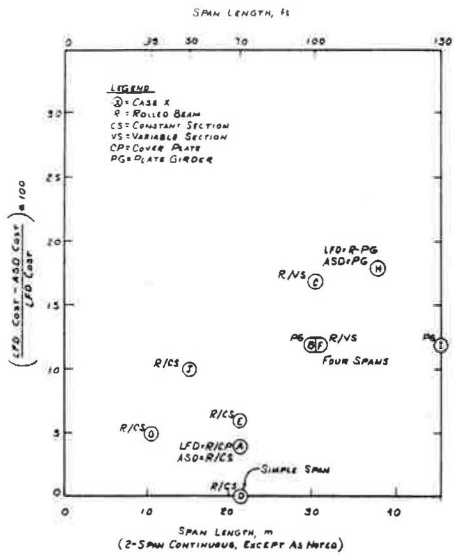
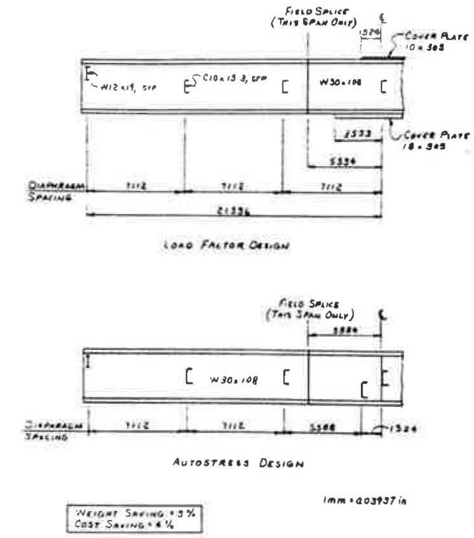


Figure 3. Case A.



addition of lateral braces and a web stiffener because a plastic hinge occurs at Maximum Load. The automoment of +108 kN-m (+80 kip-ft) causes a permanent downward deflection in the span of 4 mm (0.15 in.).

Case B

LFD and ASD plate girders with unstiffened webs are shown in Figure 4. The ASD-bridge depth is about 5 percent below the LFD depth and requires a thicker web over the interior pier. Termination of this web plate was selected to keep the web splice outside of the short unbraced length and to keep the shearing stress in the thinner web below the allowable stress. The three cross frames in the positive-bending region of the LFD bridge were replaced by diaphragms in the ASD bridge to satisfy the fatigue criterion. Cross frames are used near the interior pier to brace the bottom flange in compression; fatigue does not govern there. The ASD bridge was governed in positive bending by the Overload criteria; at the pier the bottom-flange size was dictated by local and lateral buckling requirements at Maximum Load and the top-flange area was dictated by rebar fatigue requirements at Service Load.

Case C

LFD and ASD variable-section rolled-beam designs are shown in Figure 5. The ASD bridge requires an additional brace on either side of

Figure 4. Case B.

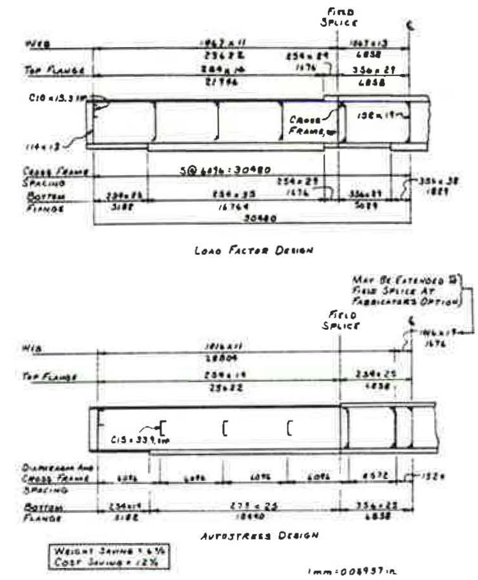
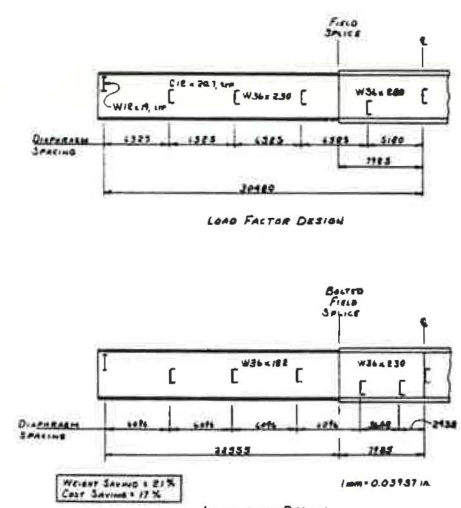


Figure 5. Case C.



the interior pier and a web stiffener at the interior pier. The ASD is mainly governed by the 0.95FY criterion at Overload in positive bending, although the inelastic pier rotation at Overload also exerts an effect on the design. The structural behavior is influenced by the relative size of the two beam sections because of the effect of stiffness on the moment distribution and because of the automoments. It was better to add material over the pier because the negative-bending region is shorter than the positive-bending region. The large cost saving for the ASD bridge is partly due to the available rolled shapes; the W36 x 230 beam is the smallest section in that series and the next smaller beam, W36 x 194, is too small. The ASD bridge does not encounter this discontinuity in the range of beam sizes.

Reason for ASD Benefits

Although any bridge has peculiar aspects, certain generalizations indicate why an ASD bridge is usually more economical than a LFD bridge. The main reason is that by using the principles of plastic design to compute the resistance at Maximum Load by mechanism formation, the size of the stringers is usually governed by Overload rather than Maximum-Load criteria; Maximum-Load criteria may however dictate flange and web proportions at a hinge location. Thus, serviceability (Overload criteria) rather than safety (Maximum-Load criteria) governs many aspects of a design. Certain details may be controlled by fatigue. The Overload criteria for ASD provide benefits in the negative-bending region by eliminating the 0.80FY limitation used in LFD to control deformations. Instead, the deformations due to negative bending are limited directly in terms of the permissible inelastic rotation. In LFD the ten-percent redistribution in moment at Overload and Maximum Load is permitted if certain compactness criteria are met. This provision, however, does not provide the same benefits as ASD because the stresses due to negative bending after redistribution are limited to 0.80FY.

In the positive-bending region of an ASD composite plate girder, the Maximum-Load positive moment is checked against the composite plastic moment in positive bending (MPP). Since this section is usually the last hinge to form, it is not necessary for the section to have the ability of rotating at MPP. However, no criteria are available for a section to just reach MPP. The LFD compactness criteria are overconservative and uneconomical in this region because they provide the ability to rotate at MPP. The current LFD criteria for noncompact sections were used in this region because the neutral axis at MPP is near or within the deck slab. The entire steel section is thus in tension so that buckling due to the bending stresses is not a consideration. The web was considered capable of resisting the relatively low shear without buckling in this region.

Future Work

Before the ASD method can be suggested to AASHTO for incorporation into their specification, certain assumptions must be subjected to additional investigation. Phase 3 of AISI Project 188 will therefore investigate, both theoretically and experimentally, the number of load cycles required for shakedown, the distribution of the inelastic rotation and concrete cracking at Overload, compactness requirements, and lateral bracing for Maximum Load (the latter two requirements, although contained in the AASHTO specification, may require modification for plastic design). Beyond Phase 3, a full-scale test subjected to actual truck loading is being planned. Development of criteria to assure that a composite plate girder can just reach MPP—particularly the limits on web slenderness—will require additional study and possibly testing.

Conclusions

The ASD method is an extension of the LFD method for steel rolled-beam and plate-girder bridges. The intention of the ASD method is to permit a more rational design of a continuous-span bridge than the LFD method; the ASD limit-state criteria are more directly related to the structural performance requirements at the three factored load levels. An additional benefit of ASD is that the resistance to Maximum Load is computed the same for simple and continuous spans—that is, by mechanism formation. Thus, the inherent strength of a continuous-span bridge can be based on the principles now used in LFD for a simple-span bridge. This is consistent with a goal of LFD to provide a uniform safety margin for live load among different bridges. ASD extends the principles of LFD toward this goal and results in weight and cost savings in the design of typical bridges.

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