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Bridge Design Procedures Based on Performance Requirements

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In 1977 the American Association of State Highway and Transportation Officials (AASHTO) introduced redundancy as a parameter in bridge design, specifically by making allowable fatigue-stress ranges a function of a structure's redundancy. However, recently proposed design procedures would allow redundancy and fatigue to be handled more directly; they would bring design assumptions in closer agreement with actual behavior. This paper reviews these new design procedures from a load-factor approach. At each factored load level, structural performance requirements are defined and then limit-state criteria are established to satisfy these requirements. The AASHTO limit states and the recently proposed limit states are compared, because both are intended to satisfy the same performance requirements. At service load, a new fatigue design procedure reflects actual conditions. At overload, the ability of a redundant structure to shake down is recognized. At maximum load, the strength of a redundant structure is computed with plastic design methods. And, finally, a preliminary procedure for fail-safe analysis that would apply only when fatigue governs is introduced. An engineer following these new procedures is encouraged to include redundancy in bridges for more rational design.

In 1977 the American Association of State Highway and Transportation Officials (AASHTO) introduced redundancy as a parameter in determining allowable fatigue-stress ranges for steel bridges (1). A new table of allowable stress ranges was added for nonredundant structures to provide increased safety by requiring a shift of one range of loading cycles for fatigue design, thereby reducing allowable stress ranges. However, research into the behavior of steel structures has resulted in new design procedures that handle redundancy and fatigue more directly. These methods close some of the gaps between design and actual conditions. Similar methods are appearing elsewhere: In California the design method is correlated with permit policy (2), and in Ontario the new highway bridge design code relates limit-state design to actual loadings (3).

This paper describes four new design procedures that use the same approach as that currently used in the AASHTO load-factor design (LFD) specifications (4). Before describing these methods, it will be helpful to review performance requirements.

STRUCTURAL PERFORMANCE REQUIREMENTS

For any loading, a designer understands the behavior expected of the bridge. This behavior may be stated in terms of structural performance requirements associated with various load levels. The requirements for the AASHTO LFD method are shown in the table below.

Source	Load Level	Requirement
AASHTO	Service load [D + (L + I)] (dead plus standard vehicles)	Provide fatigue life, control elastic deflections, limit concrete cracking
	Overload [D + 5/3(L + I)] (dead plus occasional permit vehicles)	Control permanent deformations possibly impairing ride quality
	Maximum load {1.3[D + 5/3(L + I)]} (increased dead plus few exceptional vehicles)	Resist load
Proposed	Fatigue load [(L + I) 50/72], or by study (effective truck)	Provide fatigue life
	Fail-safe load [αD + β(L + I)] (only checked if a detail has a design life less than a specified value, e.g., 100 years)	Resist load when one element is separated

To ensure that a bridge behaves according to the stated structural performance requirements, a designer may establish limit states for each requirement. A limit state is a constraint such that if the structural behavior exceeds the constraint the performance requirement may not be satisfied. Various limit states may be selected to satisfy the same performance requirements. For example, the AASHTO LFD and working-stress design methods both result in bridges that behave satisfactorily. However, a standard by which the merit of a limit state can be judged is how close that limit state is to actual behavior.

In the following section, four recently proposed design methods and their limit states are described. The methods are grouped under four load levels: the three AASHTO factored load levels and a proposed load level.

PROPOSED DESIGN METHODS

Service Load

When subjected to service loads, a structure's primary performance requirement is that it have an adequate fatigue life. AASHTO currently achieves this performance requirement by limiting allowable stress ranges.

The American Institute of Steel Construction (AISC) encouraged good design in the publication of the bridge fatigue guide by Fisher (5). Fisher's data, which were derived from constant-amplitude tests, are also the bases for the AASHTO specification. As an outgrowth of National Cooperative Highway Research Project variable-amplitude fatigue studies, Schilling and Klippstein recently proposed an extension of the current design method to recognize the actual spectrum of truck loading (6). The proposed method would overcome many of the current discrepancies between assumed and actual fatigue conditions.

For example, consider a relatively short simple-span bridge for which the fatigue design is governed by truck rather than lane loading. Assume that the average daily truck traffic is 2000 and that each passage of a truck causes one loading cycle. This traffic will cause 36 500 000 loading cycles in 50 years—a reasonable minimum life. According to the 1970 Federal Highway Administration (FHWA) nationwide loadometer survey, 7.37 percent of these cycles, or 2 690 000 cycles, would exceed a weight of 320 kN (72 000 lbf), which corresponds to the AASHTO HS20 truck. The current AASHTO specifications require that main longitudinal members under an average daily truck traffic of less than 2500 be designed for 500 000 stress cycles caused by an HS20 truck. Field measurements have shown that the actual stress ranges in bridges are usually considerably lower than those that would be calculated by AASHTO procedures. Thus, the AASHTO design conditions for this example combine an artificially high fatigue stress with an artificially low number of stress cycles.

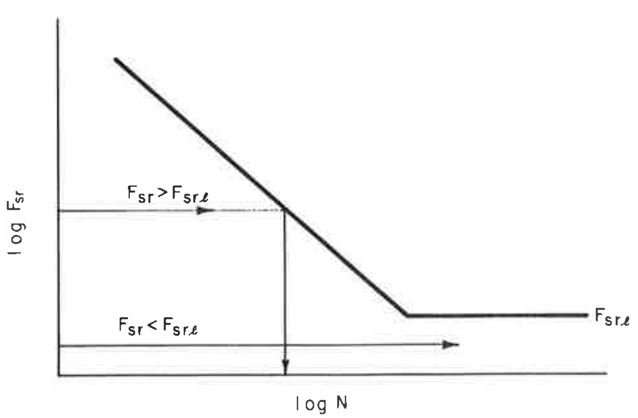
The fatigue design procedure proposed by Schilling and Klippstein is based on stresses caused by a fatigue design truck. This effective truck has the same geometry as the HS20 truck, except that its gross weight (W_F) is determined from

$$W_F = (\Sigma \alpha_i W_i^3)^{1/3} \tag{1}$$

in which α_i is the fraction of trucks weighing W_i. If information is not available on the expected distribution of truck traffic, W_F is conservatively taken as 220 kN (50 000 lbf). This loading is shown from a load-factor approach in the table above.

The design stress range caused by this truck is calculated by placing the truck in positions that cause the maximum and minimum stresses at a detail. The truck is placed in only one lane and no lane loading is considered. In calculating these stresses, lateral load-distribution factors of S11 and S7 are used for interior and exterior girders; S is the lateral spacing of girders

Figure 1. Fatigue design curve.



(in customary feet). These distribution factors are based on available experimental and theoretical information.

The maximum stress range for infinite life (F_{srl}) has been established for each of the AASHTO detail categories. If the calculated stress range (F_{sr}) for a detail does not exceed this maximum range, the fatigue life is infinite and no further fatigue check is required (Figure 1). Otherwise, the minimum estimated life of the detail must be calculated. First, the estimated minimum number of loading cycles to failure is determined from formulas or from SN curves. Next, the life in years is calculated:

$$L = N/365\ T\ P \tag{2}$$

where

L = life in years,
N = life in cycles,
T = average daily truck traffic, and
P = stress cycles per passage.

Design values of daily truck traffic are given (6) for use when better data are not available. Usually, there is one stress cycle per truck passage for longitudinal members and there are three cycles for transverse members.

The new method can be tailored to the specified values of key parameters and used to compute the remaining life of existing bridges.

Overload

The overload structural performance requirement—adequate riding quality—deals only with serviceability; ultimate capacity is not considered. The LFD method controls permanent deformations by limiting flange stress to a percentage of the yield stress. If certain compactness requirements are met in a continuous bridge, 10 percent of the negative interior-support moment may be redistributed before the stress calculations are made. No redistribution occurs, because the allowable flange stress due to negative moment is increased to a value that is still below the yield stress.

A proposed design method known as autostress design (ASD) would place no restriction on the maximum stress due to negative moment (7). Instead, the ASD method permits a continuous-span bridge to shake down, that is, to undergo small plastic deformations at a pier that will stabilize after a few cycles and produce elastic behavior. Shake-down demonstrates a benefit of redundancy by developing a set of residual forces and moments that remain when the live load is removed. These moments are termed "automoments" because they are automatically developed.

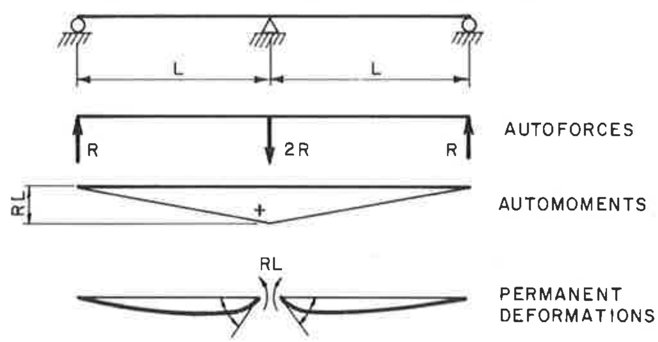
For the two-span bridge illustrated in Figure 2, the automoment reduces the interior-support dead-load moment. The AASHTO specification provisions for hybrid beams implicitly recognize the beneficial effect of autostresses by ensuring elastic behavior after initial local web yielding. In this case autostresses are equilibrated within a cross section. Automoments may be considered in an ASD bridge design, but in reality they are developed only if needed to produce elastic behavior.

Maximum Load

The maximum-load structural performance requirement deals only with load resistance; 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 the elastic moments with the same possible 10 percent redistribution mentioned above, in partial recognition of the reserve strength of a redundant structure. In the LFD method, a simple-span bridge reaches its maximum carrying capacity when a plastic hinge forms near midspan. This hinge and the two true hinges at the supports form a mechanism that prevents additional loading.

The concept of mechanism formation permitted by the LFD method for simple-span bridges could be extended to continuous-span bridges. In a two-span redundant continuous bridge, a plastic hinge usually develops first at the interior support, but this hinge is not sufficient to form a mechanism. Additional loading is required to cause a second plastic hinge at about midspan. These two plastic hinges, together with the true hinge at the exterior support, form a mechanism that prevents additional loading. During the formation of the second hinge, the interior-support plastic hinge is assumed to rotate inelastically at the plastic moment. To ensure that this inelastic rotation can occur, special limit-state criteria are required. The AISC specification (8) contains such plastic-hinge criteria for buildings, and similar criteria are being developed for bridges as part of the American Iron and Steel Institute (AISI) Project 188.

Figure 2. Typical automoment diagram for a two-span bridge.

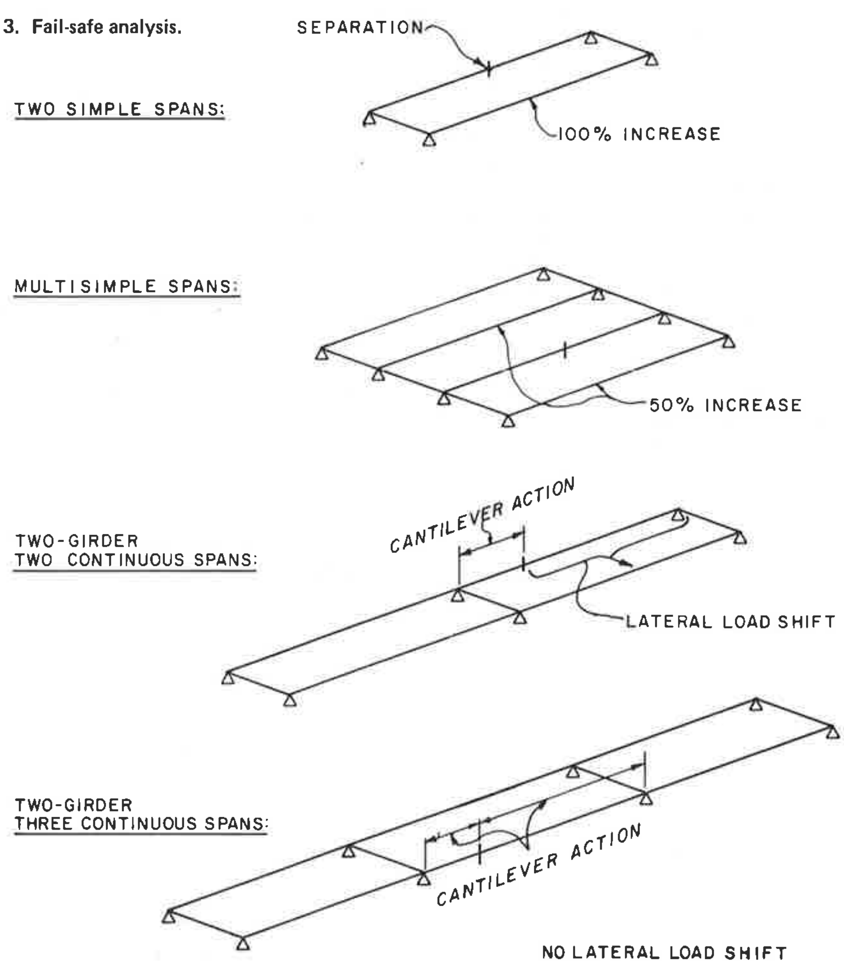


Fail-Safe Load

As mentioned earlier, AASHTO has introduced redundancy into fatigue design. Main load-carrying member components subjected to tension are considered by AASHTO to be nonredundant when failure of a single element could cause collapse. AASHTO's definition of a nonredundant structure is consistent with the conventional mathematical definition of a nonredundant, or statically determinate, structure; it is a structure in which all stress resultants and reactions can be found from equilibrium equations alone. Removal of one level of determinacy, such as a member, a reaction, or a member separation, would make part or all of the structure a mechanism.

Mathematically, the definition of a redundant or statically indeterminate structure is rather simple: Stress resultants and reactions cannot be found from equilibrium equations alone. Compatibility equations are needed for elastic design, and plasticity equations are needed for plastic design. AASHTO narrows this definition to multiload-path structures in which a single fracture in a mem-

Figure 3. Fail-safe analysis.



ber cannot lead to collapse. But questions remain: Should the reduced structure be able to carry the dead load, service load, overload, or maximum load? In calculating the strength of the distressed structure, should the designer consider elements such as curbs and railings and noncomposite slabs that in reality are partially composite? What level of damage may the reduced structure sustain in carrying the loads? AASHTO, recognizing that consideration of these questions would go beyond the designer's duty, provided some guidance in a commentary (1).

Multigirder structures (those having more than two girders) are considered redundant, which implies that the strength of two girders, simple or continuous, is adequate for a three-girder bridge. Single box girders and dual-plate girders are considered nonredundant. The commentary does not state whether a multispan indeterminate structure can always be considered redundant.

In a paper on the repair of the I-79 bridge across the Ohio River, Schwendeman and Hedgren (9) discussed the behavior of this bridge after one of the girders fractured. Although the bridge was designed as a nonredundant two-girder system, actual multiple redundancies prevented collapse while the bridge was open to traffic and before the failed girder was discovered. This incident demonstrates how valuable an asset redundancy is. To include redundancy more directly in design and thereby begin to answer the above questions, a new fail-safe check is proposed.

As shown in the table, the structural performance requirement at the fail-safe load is to provide adequate load-carrying capacity when a bridge has one separated element. The fail-safe load would only be considered when the design of a member is governed by fatigue. If the design of a bridge were governed by either overload or maximum load, a fail-safe check would not be necessary. However, if the design life of the structure were less than a certain value, say 100 years, a fail-safe analysis would be required.

The fail-safe structure would be defined by assuming a through separation at any section where the design life is less than the specified value. Only one through separation would be assumed in a fail-safe structure, so that several fail-safe structures may have to be considered. In analyzing the fail-safe structure, the load would be distributed laterally or longitudinally, as appropriate. Illustrative examples are shown in Figure 3.

Much work would be needed to make fail-safe analysis a realistic design tool. For example, the design life at which the probability of separation becomes significant should be established on the basis of statistical analyses. However, the approach offers opportunities to ensure that fail-safe structures will support specified loads.

CONCLUSIONS

A review of the structural performance requirements and the associated limit-state criteria for service load, overload, and maximum load indicates that recently proposed procedures offer opportunities to close some of the gaps between design and actual conditions. Fatigue design can be based on service-load stresses and cycles encountered during actual service rather than those calculated for artificial design conditions. Autostress design recognizes the ability of redundant structures to shake down when overloads cause localized yielding. The reserve strength of a redundant structure under maximum load can be taken into account by plastic-design procedures. Fail-safe analysis is presented as a basis for discussion and further development. These new methods encourage the use of redundant structures and permit steel bridges to be more rationally designed.

DISCLAIMER

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