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Allen, D. E.

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by D.E. Allen

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Limit states design: What do we really want?

D. E. ALLEN
Building Structures Section, Division of Building Research, National Research Council of Canada, Ottawa, Ont., Canada K1A 0R6

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The purpose of the paper is to open discussion on the development of Canadian structural codes and standards. It argues for a common simple limit states format for all civil engineering structures, with the loading function contained in structural use codes the same for all construction materials, and the resistance function contained in materials design standards the same for all structural uses. The format for loading and resistance functions and the degree of code complexity are discussed.

L'auteur veut, par cet article, susciter des échanges de nature à favoriser le progrès des règlements et normes touchant le calcul des structures au Canada. Il préconise, à cet égard, la mise au point et l'usage d'une version simplifiée et unique, fondée sur le principe des états limites, applicable à toutes les structures du génie civil et comportant, d'une part, une seule définition des sollicitations (la fonction sollicitation) peu importent les matériaux mis en œuvre, et d'autre part, une seule définition de la résistance (la fonction résistance) valable pour toutes les applications structurales. La forme des fonctions sollicitation et résistance et le degré de complexité des règlements font l'objet d'une discussion.

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Introduction

Yudcovitch (1978) has pointed out that more precise structural design methods decrease the total cost of a building project by no more than about 1% (the corresponding figure for "pure" structures such as bridges and towers is about 5%). He therefore urged that "more investigation and discussion with practising engineers be undertaken prior to any such basic modification to the code." In Great Britain (Concrete 1979) many practising engineers accept limit states design principles but feel that the refinements embodied in the recent British concrete code (British Standards Institution 1972) require more than ever the use of tables and packaged computer programs to arrive at simple answers. As practising engineers are the people most affected by changes in structural codes and standards, they should have a major influence on the format and style of these documents. This paper, therefore, opens the discussion Yudcovitch suggested by pointing out the need to replace working stress design by a common, simple, limit states format for all civil engineering structures. Discussion on the definition of this format is needed before more limit states design standards are introduced.

Working stress design

Working stress design once provided a unified approach to the design of all civil engineering structures, with a simple, common format that was easy to remember. It has been a practical tool for engineers. Is it really necessary to replace this by something else?

The method of working stress design is based on local stresses (determined by elastic structural analysis) reaching some form of material failure such as yield, crushing strength, or onset of permanent deformation. Elastic structural analysis has a practical advantage since only one theory is needed for all design calculations. Working stress design, however, runs into difficulty when the fundamental structural requirements of strength and serviceability (limit states) are examined more closely.

The first difficulty is that localized material failure frequently does not represent failure of the structure. One example is the lateral stability of masonry walls. The current masonry code (Canadian Standards Association (CSA) 1977) allows 0.25 MPa tensile stress for lateral loads, about one tenth of the allowable compressive stress. A masonry wall subjected to many years of weathering and differential movement may not have any tensile strength at all in the mortar joints. An allowable tension of 0.25 MPa, therefore, appears to be a fictional value whose purpose is to keep the force resultant sufficiently far from the edge of the wall to prevent overturning. It is derived from an earlier rule of zero tension, which keeps the force resultant within the middle third of the wall. The trouble with this approach is that it gives no indication of the lateral stability or safety factor of the wall against wind collapse. Another example is the allowable concrete tensile stress in the current rules for prestressed concrete. What is it for? Not for strength, since this must be checked by an ultimate strength calculation. Is it for crack control? If so, why is prestressed concrete treated differently from reinforced concrete?

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The second difficulty is that working stress design does not, in any consistent way, consider constraint stresses which occur as a result of manufacturing or construction processes (e.g., residual stress, prestress, and clamping forces in bolted connections), shrinkage, differential settlement, and temperature variation. Should they be included in the calculations or not? As a general rule they are omitted because they do not affect the strength of normal ductile structures. In fact, if stress concentration is included in addition to other constraint forces, then the more accurate is the stress analysis the more unrealistic and uneconomical is the structure. If the structure fails in a brittle manner, however, they should be included. If cracking or permanent deformation is a potential problem, they also should be included. Working stress theory gives no guidance on this question.

The difficulty with constraint stress also applies to stress redistribution. Sometimes failure stress (elastic stress at initial material failure) underestimates the strength of the structure. One example is the compressive strength of reinforced concrete where the yield strength of the reinforcing is added (incorrectly according to elastic stress analysis) to the compressive strength of the concrete. Another example is the post-buckling capacity of stiffened metal compression elements. Stress redistribution finally allows greater possibilities in design, for example by reducing congestion of reinforcing steel, or by the use of simple plastic theory. For continuous slabs and beams the latter is simpler and more realistic than elastic theory.

Finally, the safety factor is applied to the stress, a procedure which can occasionally be misleading. For the serviceability limit states, where the safety factor is essentially 1.0, it works quite well. For the ultimate limit states, where the stresses due to applied loads are additive, it also works quite well, although some economy could be gained by reducing the safety factor on dead load. The stress safety factor runs into trouble, however, where there are counteracting loads or constraint stresses. If the dead load stress is opposite to the wind stress, the safety factor is applied to a small difference between two large numbers, with the result that a small increase in wind load will cause failure. The probability of failure therefore becomes quite high as the wind stress approaches the dead load stress (Fig. 1), and this was a significant factor in the collapse of the Ferrybridge Cooling Towers (Allen 1969). Similarly, if prestress opposes applied load stress, a small increase in load will cause a considerable change in stress, whereas if prestress is in the same direction and large in comparison with the load stress a large increase in load will cause only a small increase in stress. To get consistent safety, therefore, the safety factors on stress should vary as a function of the algebraic ratio of prestress to

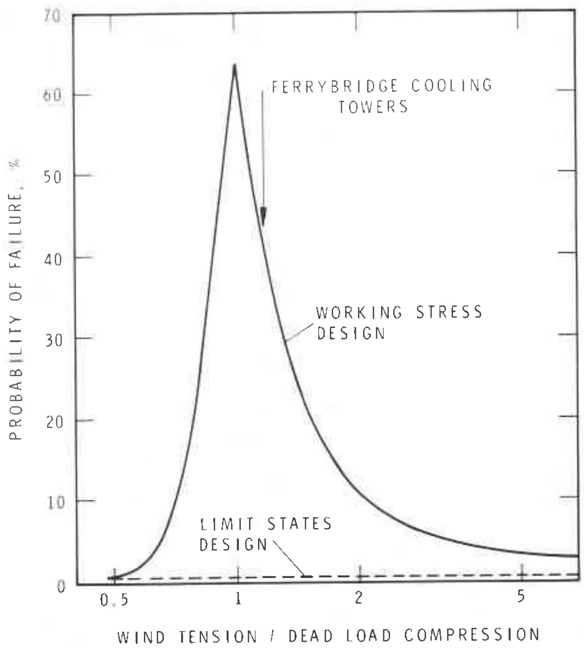

FIG. 1. Probability of failure for counteracting load effects.

load stress (Tochacek and Amrhein 1971).

Working stress design has been modified to overcome many of these shortcomings. For example, working stress formulae are generally based on member strength rather than material failure, e.g., those for steel in bending, reinforced concrete in compression or shear, and interaction formulae for combined axial load and bending. Quite often allowable stresses are not related to local (e.g., extreme fibre) stress but are member strengths expressed in terms of force divided by area, as for buckling, connectors, and truss components. For composite steel and concrete structures, the constraint stresses arising during construction are ignored. Rules have been introduced to give more consistency when considering counteracting loads and overturning. An adjustment in allowable stress could be made to gain economy for structures carrying mainly dead load. In fact, working stress design has been made equivalent to ultimate strength design (Schmidt 197?) by a number of such adjustments. Qualifications and adjustments of this kind, however, blunt working stress as a practical tool for design. Such a deficiency would not be particularly serious except for the changes that have taken place in civil engineering structural practice.

Changes in practice

Structures, particularly building structures, are becoming lighter and thinner, with less effective damping. This has arisen for a number of reasons—higher strength materials combined with reduced safety factors, composite action, and less contribution towards stiffness and damping from non-structural components. The result is that serviceability is becoming more critical in design, and the limit states of deflection, vibration, and cracking are becoming as important as strength. Sometimes stress is a useful tool for this (e.g., for crack control or yield), sometimes it is not (e.g., for deflection and vibration). Even for crack control its application is limited. Constraint stresses often make it difficult to control cracking and it might be much more effective to provide an impermeable membrane against leakage and to design the structure for strength. Also, corrosion might be better controlled by detailing reinforcement and cover than by restricting tensile stresses due to applied loads.

Fire resistance and structural integrity to contain local damage due to accidental loads are other limit state conditions not traditionally considered that may occasionally require design calculations. As very large deformations and constraint forces can arise under these "accidental" conditions, ultimate strength analysis rather than elastic stress analysis is appropriate.

Rehabilitation of existing structures is becoming much more common. For new structures under design, Yudcovitch (1978) has already pointed out that further refinement does not make that much difference in overall cost of a project. Rehabilitation costs, however, increase considerably if the existing structure is deemed to be unsafe. The problem has become particularly acute for bridges where truck loads have gradually increased over the years and, if realistic loads were used, a great number of existing bridges would be unsafe according to working stress design. For economic reasons, therefore, evaluation of existing structures requires a closer description of strength and reliability than is provided by the working stress concept.

Structures are being used for a greater variety of purposes than before: for buildings of various kinds, for bridges, tanks, retaining walls and dams, pipes and sewers, towers and poles, falsework, nuclear containment structures, and offshore platforms, to name the applications prevalent today. Thus there is a greater need to identify different limit states for different structural uses and different environmental conditions. It is also necessary to ensure that design criteria previously derived for traditional construction apply to a wider number of applications in the future.

In addition to wider application to existing structures, there is a growing number of construction types to be considered. Composite structures are being introduced, e.g., wood and steel trusses, composite timber and concrete decks, air-supported membranes, composite soil and steel culverts, and reinforced earth and stone retaining structures. There are also various mixtures of prefabricated and cast-in-place types of construction. The tendency towards greater variety of construction types and the wider application of any given type means that a unifying design basis is needed as much as, or even more than, in the past.

Lack of a unifying basis is particularly evident in the recent growth of codes and standards, which contain a rather haphazard mixture of methods and criteria. In fact, the situation is beginning to get out of control. Table 1 shows the matrix of civil engineering codes and standards. Across the top are the use codes for buildings, bridges, towers, etc., and down the side are the various materials involved in each of the use standards. As both uses and construction types as well as the experience and knowledge of each increase, the size of structural standards grows exponentially, particularly if each use code includes all criteria related to the behaviour of various construction types. The new Ontario Bridge Code (Ontario Ministry of Transportation and Communications 1979), for example, is about 1200 pages thick, including commentaries. One way to control this tendency is to organize all civil engineering structural codes and standards into material design standards applicable to a wide variety of uses and use codes containing basic requirements for all construction materials. To do this requires a unifying basis with a common terminology and common format that separates the loading side of the design criteria (contained in the use codes) from the resistance side (contained in the material design standards).

Limit states design

Limit states design is intended to fulfil the need for a unifying basis in the same way that working stress design has done in the past. In addition, limit states design has the following advantages.

(1) It gives the designer a better understanding of the fundamental structural requirements and of the behaviour of the structure in meeting these requirements. This enables him to exercise better judgement in the design and evaluation of structures used for different purposes and subjected to different environmental conditions.

(2) It provides reliabilities more consistently related to the consequences of failure, and as a result, is more economical for cases which were previously overdesigned (e.g., structures under high dead load, structures such as bridge slabs whose strength was considerably underestimated by working stress design, and structures whose failure does not result in serious consequences), and better safety for those rare cases previously underdesigned (e.g., components subject to counteracting loads). It therefore results in an overall material economy.

TABLE 1. Codes and standards for civil engineering structures. X means that a code or standard applies

	Buildings	Bridges, culverts	Retaining walls, dams	Pipes, sewers	Towers, poles	Falsework	Nuclear	Offshore
Concrete	X	X	X	X	X		X	X
Steel	X	X	X	X	X	X	X	X
Aluminum	X	X			X	X		
Timber	X	X	X		X	X		
Masonry	X	X	X	Not any more				
Ground	X	X	X	X	X	X	X	X
Air-supported membranes	X					X		
Glass	X							

TABLE 2 . Limit states terminology

Term	Symbols	Definition or examples
General		
Limit state		A specific form of failure
Ultimate limit states		Failures affecting safety
Serviceability limit states		Failures affecting use or durability
Specified value	f_c', f_s', D', L'	Specified in codes, drawings, etc.
Partial factor	$\alpha, \psi, \gamma, \phi$	Load factor, resistance factor, etc. (see below)
Factored value	f_c, f_s, D, L	Specified value × partial factor
Loading function		
Load (action)	D, L, A	Applied force or imposed deformation
Load factor	α	Uncertainty in loads and load effects
Load combination factor	ψ	Reduced likelihood of loads acting together
Importance factor for use	γ	Adjustment for consequences of failure
Load effect		Internal force, stress, deflection, etc.
Resistance function		
Resistance (strength)	f, R (or subscript r)	See [5]–[7]
Resistance factor		
Material	ϕ_c, ϕ_s, etc.	Uncertainty and behaviour of material failure mode
Member	ϕ	Uncertainty in resistance formula or dimensions
Nominal resistance	R'	Resistance without resistance factors

(3) It has been adopted by the International Standards Organization as the basis for international model standards. This will be important for Canadian engineers involved in projects outside Canada.

In fact, limit states design has been gradually introduced over many years, first for concrete and more recently for steel, and is now being developed for other materials. The problem is to give it the definition it needs to make it a practical tool for the design of all civil engineering structures and also serve as a basis for harmonization of codes and standards. A CSA technical committee representing each of the codes and standards affected (originally a CSA/National Building Code joint committee) has been set up to do this, and a set of guidelines (CSA 1981) drawn up containing basic principles, common terminology, and format. The common terminology is given in Table 2; the basic format is as follows.

Ultimate limit states:

[1] factored resistance $\geq$ effect of $\boxed{\text{factored loads}}$

Serviceability limit states:

[2] $\boxed{\text{serviceability criterion}}$ $\geq$ effect of $\boxed{\text{service loads}}$

The solid boxes (loads, load factors, and basic serviceability criteria) are generally contained in the use codes and the hatched boxes (resistances, performance factors, and methods of analysis) are contained in the material design standards.

The format for serviceability limit states, [2], is basically unchanged from that of existing standards, but the format for the ultimate limit states requires closer study, prticularly the loading and resistance functions.

Format for loading

Traditionally, loads have been classified as dead loads, D, which act permanently, and live loads, L, which are expected to vary during the life of the structure. Loads that are not likely to occur, but which may have to be considered as significant in the design (vehicle impact, explosion, and fire), are called accidental loads, A. The criteria for combining loads for design calculations depend on the frequency and duration of action of each of the loads.

The present load format for limit states design in the National Building Code of Canada (1980) and Ontario Bridge Code (Ontario Ministry of Transportation and Communications 1979) is

$$[3] \quad \text{factored loads} = \Sigma\alpha_i D_i + \psi\Sigma\alpha_j L_j$$

where the load combination factor, ψ, is equal to or less than 1.0, depending on how many live load items are being considered. The National Building Code considers 4 load items, with about 6 or 7 significant combinations. The Ontario Bridge Code considers 24 load items, with 17 load combination cases. For any particular structure, however, the number of combinations requiring consideration is usually much smaller.

Another way of looking at load combinations is to combine the maximum lifetime value of the predominating live or accidental load with the frequent or sustained values of the other loads (Turkstra's principle):

$$[4] \quad \text{factored loads} = \Sigma\alpha_i D_i + \alpha_1 L_1 + \Sigma\alpha_j L_j'$$

where L_j' refers to a frequent or sustained value of the load. One advantage of this approach is that it provides a principle for determining load combination rules that is useful to designers when considering situations not covered by codes. The frequent or sustained values of the loads may also be appropriate for serviceability calculations, e.g., creep deflection. For practical code applications, [4] may be simplified to include only those cases that are significant for a particular use. For example, Ellingwood *et al*. (1980) used this principle to derive a loading format for buildings in the United States.

The guidelines (CSA 1981) give a linear transformation with a load factor matrix, α_{ij}, as a general rule[1] and suggest [3] and [4] as a basis for simplification. A simple load combination rule seems to be appropriate for most structures (see section on code complexity), but Turkstra's principle, stated in words, would also guide the designers for nonstandard cases, e.g., industrial buildings. Should there be a common format for the loading function for different structural uses and, if so, which one is best?

Format for resistance

Working stress design has a very simple format for resistance, namely the stress corresponding to some form of material failure divided by the stress safety factor. The stress safety factor takes into account differences in material failure modes, including differences in their variability, but not differences in the loads or peculiarities of load combinations.

When plastic design was introduced, the emphasis was changed to strength, and the safety factors were applied to the maximum expected loads and therefore called load factors. As the safety factors were applied to the loads they could take directly into account the differences in loads and peculiarities of load combinations, but could not differentiate between material failure modes. The disadvantage is that different sets of load factors would have to be specified in the materials standards for different material failure modes, and this is not a practical scheme. The steel code solved this problem by multiplying the allowable stresses given in the allowable stress section of the standard by the safety factor for yield, 1.7; this gives a good estimate of yield strength but not of buckling or of connection strength. Since it was decided that the concrete code would completely replace working stress design by ultimate strength design, it was necessary to introduce a resistance factor to take into account different failure modes.

It is very much in the interest of design practice and the writing of codes, standards, and handbooks to separate the loading function from the resistance function. Limit states design therefore defines load factors to take into account safety considerations not dependent on material or type of construction, and resistance factors to take into account safety considerations not dependent on loading or structural use. In Europe, and for composite structures in the Canadian steel standard (CSA 1974), the resistance factors are applied to the materials or connecting devices (material factors). But for concrete design in the United States and Canada (CSA 1973), the resistance factors are applied to the member strengths (member factors). The question arises—should there be a common approach for different materials and types of construction and, if so, which method should be adopted?

For homogeneous structures (i.e., made of one material only) it does not matter, since the member resistance, R, is determined as follows:

[1]For accidental situations (e.g., fire, impact, or loss of support) the load factors are reduced to 1.0. This results in approximately the same overall risk of failure.

$$[5] \quad R = \beta A f$$

where f is the material resistance, A is the area (or other geometric parameter such as section modulus), and β is a reduction factor for buckling (often equal to 1.0). For a composite structure made of two or more substantially different materials, however, there is a difference, since member resistance, R, is determined as follows: compression and shear (parallel behaviour),

$$[6] \quad R = \beta_1 A_1 f_1 + \beta_2 A_2 f_2$$

bending (weakest link behaviour),

$$[7] \quad R = \min\,(\beta_1 A_1 f_1,\ \beta_2 A_2 f_2,\ A_{\mathrm{conn}} f_{\mathrm{conn}})$$

where the subscript "conn" refers to a connecting device. Sometimes the strength of a connecting device or anchor itself is given by min $(A_1 f_1, A_2 f_2)$.

A disadvantage of the member factor for composite structures is that it does not take into account the differences in variability and behaviour of the different material failure modes within a structural member. An example of this is a concrete-filled steel pipe column whose strength is determined from [6]. If the pipe column is thick, short, and has a thin steel enclosure, then the strength is governed primarily by the concrete, and the member factor would have to correspond to that for concrete in compression. If the member is short and thin and a thick shell pipe is used, then the strength is largely determined by the yielding of the steel, and the member factor is consequently much higher. To get around this difficulty the member factor should vary as a function of the percentage of steel. It is not this simple, however, because as the length of the column increases, the resistance to buckling of the concrete component decreases much more rapidly than does that of the steel component. The member factor should therefore vary as a function of slenderness as well as percentage of steel. If for simplicity a constant value is used for the member factor, then a penalty in material consumption occurs for members whose resistance is governed primarily by the steel component. Other examples where the same principle applies are shear, anchorage, and combined bending and compression in reinforced concrete structures, and the difference between friction and cohesion in soils.

This disadvantage does not occur with the use of the material factor because each material failure mode of the composite structure is assigned its appropriate resistance factor and there is no need for varying factors. (Incidentally the material factor is one advantage of working stress design.) The reasoning is the same as for the separation of dead load and live load factors.

There are, however, safety considerations related to member resistance though not to material failure mode, e.g., uncertainty in the member resistance formula, dimensional variations, and member importance. The guidelines (CSA 1981) have therefore introduced a member factor in addition to a material factor. For code simplicity the member resistance factor, generally around 0.9–1.0, might be incorporated into the resistance formula.

Because of the increasing use of composite structures containing different materials and connecting devices, it appears that such a common resistance format is desirable. The member factor has been used in concrete practice for many years, and it is therefore important that practitioners express their opinions on this question before code decisions are made.

Code complexity

A scientific understanding of structural behaviour and reliability is all very well, but to what extent should it be applied in practice? Yudcovitch (1978) provides one way of looking at the question, namely the economic return for increased accuracy. This includes time spent by the designer in pondering unfamiliar formulae and making calculations (possibly alleviated by the computer, but even here there is "turn around time"). Economic considerations indicate that modelling precision is not needed for designing new buildings, particularly small ones, but that more precision is needed for "pure" structures such as bridges and for key members, and particularly for evaluation of existing structures. Also, some structures are "one-off" and designed by one engineer without the aid of computers, handbooks, etc. (it would take too long), whereas others are mass-produced and therefore justify greater design precision. These considerations indicate a need for some flexibility.

Failure statistics also shed some light on this question. Available information (Hauser 1979) indicates that human error is the predominating influence in most failures, and that safety and serviceability risk levels are controlled and adjusted mainly through quality assurance procedures (checking, inspection, etc. to counteract errors) rather than by adjusting the safety factors. This suggests that a simple design format with relatively few numerical safety factors is appropriate for most civil engineering structures. (More complicated formats can, in fact, lead to greater likelihood of human error.) Vastly different risk levels are required, however, for different structural uses, ranging from farm storage sheds at one end of the scale to nuclear containment structures at the other. In addition to the differences in quality assurance procedures required for different risk levels, therefore, there is also a need for some flexibility in modelling precision.

How is this flexibility to be achieved? It seems from the above that it should primarily be in the estimation of loads and resistances, not in the basic format and safety factors. One clause that provides flexibility is that for axial load and bending in the concrete standard (CSA 1973). It states the fundamental principles and leaves it to the designer to take it as far as he wants. He can use handbooks and computer programs or he can easily recall a very simplified form of the principle. The latter is particularly useful for preliminary design and checking. A clause written in the form of principles also eliminates the need for an explanation of its meaning in a commentary.

If the resistance requirements are stated by a variety of formulae in which the fundamental principles are not self-evident, it is much more difficult to do this. For this reason $\sqrt{f_c'}$ and $\sqrt{F_y}$, which appear in concrete and steel standards, might well be replaced by parameters with clearer physical meanings. In shear and torsion formulae for concrete structures, for example, $\sqrt{f_c'}$ might be replaced by the tensile strength of concrete, f_t. In buckling formulae for steel structures, the idea of critical slenderness, defined as the slenderness at which the elastic buckling strength is equal to the yield strength, might be introduced into the standard, i.e.,

$$F_y = F_E = k^2E/(\text{critical slenderness})^2$$

or

$$\text{critical slenderness} = k(E/F_y)^{1/2}$$

where the coefficient k takes into account the kind of buckling under consideration, e.g., column buckling ($k = \pi$), shear buckling, local buckling, and lateral-torsional buckling. The buckling coefficient, β in [5], is then given directly in terms of the ratio of slenderness to critical slenderness (see the column formula in the steel standard (CSA 1974)). Slenderness limitations, for example for flanges in compression, can also be expressed in terms of critical slenderness instead of as a constant divided by $\sqrt{F_y}$. Expressions containing $\sqrt{f_c'}$ and $\sqrt{F_y}$ also suffer because they depend on the units chosen for f_c' and F_y.

One of the advantages of working stress design is that there are *no factors* to be considered (load, resistance, and importance factors), i.e., the designer is sure that they have already been included. One way to eliminate the factors is to include factored loads, i.e., ultimate loads and service loads, in addition to specified loads, and factored material resistances in addition to specified resistances, in structural codes and standards. Should such a scheme be considered?

Conclusions

A simple limit states design format is suggested for future structural codes and standards, with resistance and loading clauses in the form of principles and varying degrees of precision given in appendices or other documents. To avoid safety factors whose values jump or vary from one mode of failure to another, a material factor is needed for resistance.

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