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Safety Factors for Stress Reversal *)

Facteurs de securité pour des contraintes variables

Sicherheitsfaktoren für Spannungswechsel

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Safety factors at present used in the design of structures (usually embodied in the allowable stresses) have been empirically derived on the basis of trial and experience. When new forms of construction come into use it cannot always be stated with certainty that existing rules will ensure adequate safety. In other cases it may be that the old rules provide too much safety and therefore are uneconomical. An important criterion in the design of structures, therefore, should be that of consistent safety, that is, the probability of failure for a given type of construction should not change when it is subject to different loads of the same type, such as dead, floor, and static wind loads, or these loads in combination.

This investigation shows that the design rules given in many building codes are sometimes unsafe for structural members subject to stress reversal. Stress reversal occurs at a critical section of a member when loads from different sources counteract each other, such as in a truss member which is in tension under dead load but undergoes compression due to wind load or nonuniformly distributed snow load. The explanation of the lack of safety against failure is simply that the design rule in which the usual safety factor is applied to the difference between two independent load effects of the same magnitude, is unsatisfactory. Although this practice is avoided by some designers, it is overlooked by many building codes.

In this discussion, code rules for stress reversal are investigated and compared on the basis of probability of failure. Calculations are limited to a critical section of a statically determinate structure subject to dead load and wind

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effects only. Although this and other simplifying assumptions are made for the calculation of failure probabilities, it is considered that the significance of the results remains valid.

Existing Design Rules

For a critical section of statically determinate structure, design rules can be expressed in the form:

$$R_0 \geqq F\,S_0, \tag{1a}$$

where R_0 is the design strength that is required,
S_0 is the design load effect, and
F is the factor of safety.

Both R_0 and S_0 are expressed in terms of the primary force causing failure, e.g. tension or bending. When the critical section is subject only to dead load and to wind effect opposite to the dead load effect (stress reversal), the design negative strength R_0^- is governed by:

$$\phi\,R_0^- \geqq D_0\,(F_w\,k - F_D), \tag{1b}$$

where D_0 is the design dead load effect,
$k\,D_0$ is the design wind effect,
F_w and F_D are load factors for wind and dead load respectively, and
ϕ is a reduction factor for material or structure.

In examples that follow, a critical section is designed according to the rules given in the following building codes: National Building Code of Canada (NBC 1965) [1], AISC Specifications for Structural Steel in Buildings (AISC 1961) [2], ACI Building Code (ACI 1963) [3], Recommendations for an International Code of Practice for Reinforced Concrete (CEB 1964) [4]. Table I lists the corresponding design rules in the form of Eq. (1b) for the following cases.

Structural Steel. Design rules for the cases of simple bending, tension, and compression of short members, are given in Table I according to NBC 1965 and AISC 1961. It will be shown from the results that a particularly important case for stress reversal in steel structures is a long member subject to buckling.

Reinforced Concrete. Critical cases for stress reversal in reinforced concrete structures occur when the negative strength is governed mainly by the reinforcing steel alone. Examples are axial or membrane stress and bending of under-reinforced sections. Design rules for reinforcing steel or for bending strength of reinforced concrete beams are given in Table I in accordance with NBC 1965, ACI 1963, and CEB 1964.

Table 1. Design rules for stress reversal

Structural Steel (Tension, Bending, Compression of Short Members)	Reinforced Concrete (Tension, Bending)
Existing Design Rules	
Working stress and plastic design *NBC 1965 (Section 4.1)* $R_0^- \geqq 1.67\,(k-1)\,D_0$ *NBC 1965 (Sec. 4.6); AISC 1961* $R_0^- > 0.75 \times 1.67\,(k-1)\,D_0$	Working stress design *ACI 1963; NBC 1965* $R_0^- \geqq 1.5\,(k-1)\,D_0$ Ultimate strength design *ACI 1963* $0.9\,R_0^- \geqq (1.1\,k-0.9)\,D_0$ *NBC 1965* $0.9\,R_0^- \geqq (1.35\,k-0.9)\,D_0$ *CEB 1964* $R_0^-/1.15 \geqq (1.40\,k-0.9)\,D_0$
Proposed Design Rule	
$R_0^- \geqq 1.67\,(k-0.5)\,D_0$	$0.9\,R_0^- \geqq 1.8\,(k-0.5)\,D_0$

Expected Loads

Dead Load Effect

For this investigation the expected dead load effect, D, is assumed to follow the normal distribution with the following statistical parameters (see reference [4], p. 6).

	$\bar{D}/D_0$	V_D
Steel construction	1.00	0.05
Reinforced Concrete	1.00	0.08

where $\bar{D}$ is the average of D and V_D its coefficient of variation.

Wind Effect

Except for large or flexible structures, the wind effect corresponds approximately to the static load due to the maximum gust. On the basis of gust measurements at Ottawa Airport, Canada, the expected wind effect, W, is assumed to follow the Extreme Value Distribution, Type 1, with the following statistical parameters:

$$\bar{W}/W_0 = 1.10 \quad \text{and} \quad V_W = 0.194.$$

For the derivation of these values it is assumed that the design wind effect, W_0, corresponds to the 30-year return period [1] and that the design life of the structure is 30 years. Gust records at other locations in Canada give values of $\bar{W}/W_0$ and V_W similar to the above.

Expected Strengths

The failure condition assumed for calculation corresponds to the yield point of steel; strain hardening and other effects such as corrosion and fatigue are therefore neglected. This assumption corresponds to the plastic moment in steel and is also used herein for reinforced concrete in tension and bending (under-reinforced section). Expected strengths of a critical section also depend on the geometry of the cross-section; except for thin reinforced concrete members in bending, however, geometric deviations are small compared to deviations in the yield point of steel.

On the basis of test information from Lehigh University [5] the expected yield point R_y of structural and reinforcing steel is assumed to follow the normal distribution with the following statistical parameters:

	$\bar{R}_y/R_{y0}$	V_{R_y}
Strength under dead load	1.072	0.0988
Negative strength during wind effect (stress reversal)	1.18	0.0900

The derivation of these values takes account of the effect of rate of straining on the yield point of steel (see reference [6], Fig. 9).

Calculation of Probability of Failure

Given the probability distributions of the expected loads and strengths, and also the design rules (including safety factors), the probability of failure can be calculated using probability calculus as developed for structural engineering by Freudenthal [7] and others. The probability of failure is herein restricted to that for the critical section due to stress reversal only.

If the loads and strength are assumed to be statistically independent, the probability of failure is given by an equation of the same form as Eq. (2.15) of reference [7]. This was integrated numerically with the aid of Simpson's Rule.

Cases Considered

In practice it often occurs that although there is no necessity to design a critical section for stress reversal, there may be inherent negative strength capacity. Cases representing different ratios of the minimum design negative strength, R^-_{0min}, to the design positive strength, R^+_0, are considered as follows:

1. $R^-_{0min} = 0$. Examples: reinforced concrete beams or unreinforced concrete elements; uplift and overturning of structures not tied down.

2. $R^{-}_{0min} = 0.03\,R^{+}_{0}$. Example: reinforced concrete shell containing the minimum reinforcement required by the ACI Building Code (0.2 per cent for 40 k.s.i. steel).
3. $R^{-}_{0min} = 0.2\,R^{+}_{0}$. Example: long steel members relatively weak in compression.
4. $R^{-}_{0min} = R^{+}_{0}$. Examples: horizontal steel I or WF beams; short steel members whose joints are as strong as the member themselves.

The results, shown in Fig. 1 for structural steel and in Fig. 2 for reinforcing steel, express the probability of failure of the critical section as a function of k (design wind effect/design dead load effect).

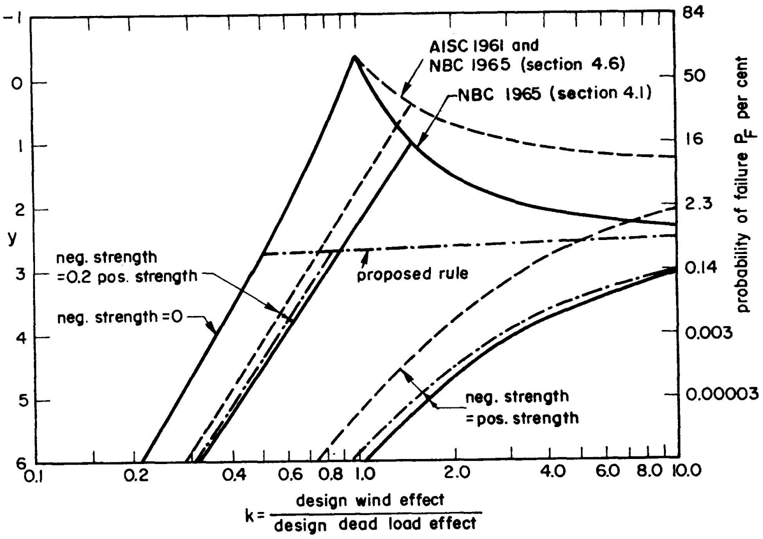

Fig. 1. Failure probability for structural steel: stress reversal (wind effect opposite to dead load effect).

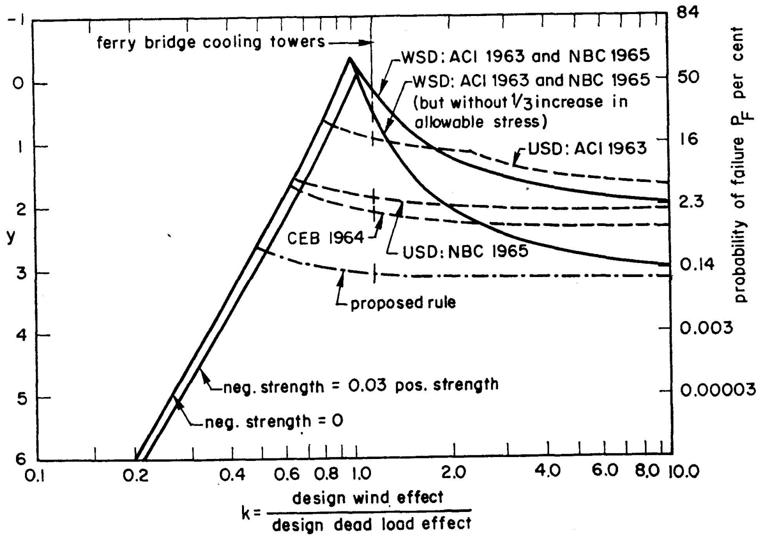

Fig. 2. Failure probability for reinforcing steel: stress reversal (wind effect opposite to dead load effect).

Discussion of Results

Figures 1 and 2 show that many existing design rules are unsafe when the negative strength is small compared with the positive strength and when k is near to one. Failure probabilities up to 64 per cent occur when the negative strength is zero and $k = 1$. This result is to be expected since most existing design rules are in error because they apply the usual safety factor to the difference between two independent load effects of the same magnitude.

Fig. 2 shows that for stress reversal ultimate strength design (USD) load factors provide more consistent safety than working stress design (WSD). It appears, however, that some revision of the load factors for stress reversal is badly needed.

Existing design rules for stress reversal in structural steel (which are the same for both elastic and plastic design) are unsafe when the negative strength is small compared with the positive strength (Fig. 1). This occurs in practice for long steel members weak in compression, or joints weak in tension. Although the results given in Fig. 1 are based on statistical parameters and safety factors applicable only to yielding of steel, they also apply approximately to buckling because the basic design rule for buckling gives roughly the same safety as the one for yielding or plastic moment.

Recommendations

One way to avoid such inconsistent safety is for the designer to apply probability calculus directly to check the probability of failure. In most cases, however, there is neither time nor sufficient information to do this. A rule of thumb for stress reversal that corresponds to the one for overturning given in NBC 1965 (Article 4.1.4.3.) is as follows: apply the usual safety factor (1.5 to 2.0) to the design wind effect minus half the dead load effect (Table 1). Figures 1 and 2 show that for the conditions assumed for calculation, this rule gives consistent safety for different values of k.

Another method of providing more uniform safety for stress reversal is to adopt a system of partial safety factors similar to that proposed in CEB 1964. (This is partially done for ultimate strength design in ACI 1963, although erroneously for stress reversal.) The advantage of the CEB system of safety factors is that it induces the designer to think more clearly because it separates the safety factor into two components, one pertaining to the load effects (load factors) and the other pertaining to the material or structure. When dead load helps prevent failure (stress reversal, overturning, etc.), then a load factor less than 1.0 should be used.

Collapse of Ferrybridge Cooling Towers

If the situation is as serious as is indicated in Figures 1 and 2, it is surprising that it has not been corrected. There are, however, a number of reasons why there have not been more failures:

1. Some structures have as much negative strength (stress reversal) as positive strength. (See Fig. 1: negative strength = positive strength.)
2. Many structures are statically indeterminate and, because of stress redistribution, failure probabilities are not as severe as indicated in Figs. 1 and 2.
3. Most structures built in the past have been heavy so that k is not the critical range.
4. Designers have intuitively provided more strength for stress reversal than required by building codes.

In an age of changing technology these reasons are no longer reliable; as reported by a Committee of Inquiry [9], the collapse of the cooling towers at Ferrybridge, England was partly due to incorrect design rules based on working stress hypothesis. On 1 November 1965 three cooling towers – reinforced concrete shells — collapsed as a result of excess vertical tensile stress in the shell. The value of k for the design of these tower shells [9] at the location of failure was about 1.18. Fig. 2 shows that the probability of failure was 41 per cent according to working stress design in ACI 1963, and close to this according to British design rules [8]. The high failure probability and an error in the estimation of the wind effects appear to have been the main resaons for the failure under wind speeds with a recurrence period of only two to five years [9]. The Committee of Inquiry recommended adoption of CEB 1966; this would result in a calculated failure probability for a Ferrybridge tower of 1.7 per cent (Fig. 2).

Concluding Statement

The collapse of the Ferrybridge cooling towers and the results of probability calculations (Figs. 1 and 2) are evidence that a revision of existing design rules for stress reversal is needed. Two approaches to revision have been suggested in this discussion under the heading "Recommendations".

List of Symbols

D	dead load effect
F	factor of safety
k	design wind effect / design dead load effect
P_F	probability of failure of a critical section

R	strength
R^+	positive strength
R^-	negative strength
R_y	strength as defined by the yield point
S	load effect
V	coefficient of variation
W	wind effect
y	parameter defined by $P_F = \frac{1}{\sqrt{2\pi}} \int_y^\infty e^{-\frac{x^2}{2}} dx$
ϕ	reduction factor for material or structure
R_0, S_0, D_0, etc.	design values of R, S, D, etc.
$\bar{R}$, $\bar{S}$, $\bar{D}$, etc.	expected average values of R, S, D, etc.
V_R, V_S, V_D, etc.	expected coefficients of variation of R, S, D, etc.

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Summary

Probability calculations for statically determinate structures subject only to dead load and wind effects show that existing design rules for stress reversal are sometimes unsafe. A committee of inquiry concluded that the collapse of the Ferrybridge cooling towers was to a considerable extent due to adoption of existing working stress design rules for stress reversal. Changes in the design rules are proposed which give more uniform safety.

Résumé

Les calculs de probabilité dans le cas d'ossatures statiquement déterminées soumises uniquement aux charges propres et au vent montrent que si les contraintes changent de signe les règles usuelles de calculs n'assurent pas toujours une sécurité suffisante. Une comité d'enquête a conclu que l'effondrement des tours de refroidissement de Ferrybridge était dû essentiellement à l'emploi des normes usuelles concernant le changement de signe des contraintes. L'auteur propose une modification de ces normes afin d'assurer une sécurité plus uniforme dans tous les cas.

Zusammenfassung

Wahrscheinlichkeitsrechnungen für statisch bestimmte Tragwerke, welche nur durch Eigengewicht und Windkräfte belastet sind, deuten darauf hin, daß derzeit bestehende Berechnungsweisen für die Spannungsumkehr zu ganz ungenügenden Sicherheitsfaktoren führen können. Es ist bekannt, daß das Versagen der Kühltürme Ferrybridge hauptsächlich auf Verwendung von bestehenden Berechnungsweisen der Spannungsumkehr im Betriebsspannungsbereich zurückzuführen ist. Es werden Änderungen der Berechnungsweisen vorgeschlagen, um eine einheitlichere Sicherheit zu erlangen.