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Transfer of Australasian bridge design to fatigue verification system of Eurocode 3

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ABSTRACT

The fatigue design of steel structures given in Eurocode 3 has gained more and more significance within the engineering society. That is not only true for Europe, where it was developed, it has also received worldwide recognition and application. The system is consistent with many recent design codes in various areas, such as for cranes, off-shore structures and shipbuilding. The main basis for the fatigue regulations is the IIW Fatigue Design Recommendations, which were established by an international body with the widest possible collaboration of all relevant countries. A code does not stand alone, it is bound into a network of neighbouring codes and so, an introduction of a new code can only be made step-by-step with a reasonable time for transition. The transition from the verification procedures of the Australasian code to that of the Eurocode implies an adaption of the loading system. This should not imply a change in the loading, but an adaption in terms of the format, so that the fatigue verification procedures of Eurocode may be applied while the load models can be maintained. The paper provides an overview of the Eurocode system and puts forward a proposal for adaption of the loading system to the first harmonized Australian and New Zealand design standard for steel and composite bridges AS/NZS 5100.6.

1. Introduction

The importance of a field of engineering considerations can be derived from the statistics of damage of steel structures. The causes of damage can be a pure overload that had not been considered in design, a failure due to elastic stability problems or a fatigue failure by crack initiation and propagation. The issues of brittle fractures are becoming more important again, since cheaper steels of improper qualities are readily available on the world market. The latter can be overcome by a detailed control of chemical composition, the production process (rolling, heating, cooling, etc.) and mechanical properties. The rest of the damage causes are environment effects such as corrosion and unforeseen events such as movements and deformations caused, for example, by displacement of abutments or by seismic events. A breakdown of the different causes of damage to steel structures is presented in Table 1 [1].

As can be seen from Table 1, the two clear causes from damage in steel and composite bridges are fatigue and corrosion. The present task is related to a transfer of fatigue regulations. Corrosion protection is another, but no less important area of consideration, except in coastal regions, which is outside the scope of this paper.

In Australia and New Zealand, the bridge design loadings are presented within AS 5100.2 [2,3]. This paper presents the basis for the proposed new fatigue rules within the forthcoming steel and composite bridge design standard AS/NZS 5100.6 [4], which are based on the ideas of BS EN 1993-1-9 (Eurocode 3) [5].

2. Relevant effects on fatigue

The process of a fatigue failure consists of three phases: firstly, the crack initiation; secondly, the crack propagation; and, thirdly, the rupture of the remaining smaller section due to overload. Most of the life of a welded structure is spent in phase two. The under-critical extension of a crack is governed by the input of surface energy into the growing crack surface by a release of elastic energy of the surrounding of the crack tip. The leading material parameter is the modulus of elasticity, which is almost equal for all steels and so, all welded steel structures basically have the same fatigue properties. The use of high strength steels is only of advantage when the static strength is the limiting factor of

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Table 1
Damage statistics of steel structures [1].

Causes	Buildings (%)	Bridges (%)	Cranes (%)
Overload	33.6	14.8	36.0
Stability	20.4	8.6	12.6
Fatigue	2.6	38.3	31.5
Movement	8.2	1.6	15.3
Deformation	4.6	0.8	0
Brittle fracture	3.0	3.9	0.9
Corrosion	19.4	32.0	0.9
Temperature	7.6	0	0
Others	0.7	0	2.7
Sum	**100**	**100**	**100**

design and not fatigue. Another limiting factor is toughness which influences the size of the final crack. That must be considered at inspection intervals.

The main points in fatigue design are stress and number of stress cycles on the action side. On the side of resistance, there is the stress rising notch effect, which may be originated by design or by weld imperfections as, for example, from inclusions, pores, lack of penetration and lack of fusion. In practical design, residual stress, caused by welding, erection, displacement of abutments or seismic events has to be considered. That is done by the so called delta sigma concept (i.e. only the stress range $\Delta\sigma = \max\sigma - \min\sigma$ is taken into account, since the maximum stress at stress peak will most probably be as high as the yield stress f_y and so, the highest applied load stress range will reduce from yield). The fatigue properties and the effective stress ratio $R_{\text{eff}} = (f_y - \Delta\sigma) / f_y$ are, thus, independent from the stress ratio calculated from loads.

The notch effect can be influenced and modified by the proper choice of the structural detail in design and by the reduction of stress raising imperfections in the fabrication shop, of which the latter is achieved by appropriate quality control (see Ref. [6–10]).

3. Verification scheme of Eurocode 3

3.1. Partial safety concept

In the partial safety concept two safety factors are applied: one for the loads, covering the uncertainties of the loads; and one for the uncertainties of the resistance of the structure [11,12], including material law and modelling of the structure. Table 2 provides an overview of the system. Loadings, called actions (index S), and strength, called resistance (index R), are clearly separated. There are partial safety factors γ_F for actions and γ_M for resistance values. The partial safety factors are always bigger than unity, therefore factoring on the action side means multiplication, whilst factoring on the resistance side means division.

Values are given by characteristic values (index k). Since the data are scattered, the data for a 95% survival probability are given (or mean minus two standard deviations), of which the difference is more historical and insignificant for engineering applications. The design (index d) values are computed by factoring the characteristic values by the appropriate partial safety factors. Then the design values of the actions and the resistance are directly compared for a verification, as follows:

$$\sigma_{S,k}\gamma_F = \sigma_{S,d} \leq \sigma_{R,d} = \frac{\sigma_{R,k}}{\gamma_M}. \tag{1}$$

It is vital to distinguish clearly between a characteristic or a design value, and what is an action or a resistance value. Material properties from standards are considered as characteristic values. Unfortunately the use of the indices in the Eurocodes are not consistent in this respect. There is a variability in all data and values, which enter in a verification procedure. Each one of the different effects is associated either to γ_F or to γ_M, as shown in Table 2.

Table 2
System of partial safety verification.

Characteristic values of actions F_k	Characteristic values of resistance data M_k
multiply with	divide by
partial safety factor γ_F	partial safety factor γ_M
If applicable a combination factor ψ	
resulting in design values (Index d)	
of actions F_d	of resistance M_d
followed by	
establishing of combinations of actions	defining of limit states
Calculation or definition of	
sectional forces S_d or stresses $\sigma_{S,d}$	limit state resistance forces R_d or stresses $\sigma_{R,d}$
Verification by comparison	
$S_d \leq R_d$ or $\sigma_{S,d} \leq \sigma_{R,d}$	

3.2. Grid of S–N curves

The applied stress versus endurance cycle relation in terms of fatigue is usually represented by an S–N curve, which consists of three separate areas. At low cycles and cyclic plastic strains there is the area of low cyclic fatigue, which is not relevant in most steel structures, except for the case of seismic loads and pressure vessels. At higher cycles between about several thousand cycles and several million cycles there is a straight endurance line in the log–log plot, which can be described by the Basquin formula:

$$N = \frac{C}{\Delta\sigma^{\text{m}}} \tag{2}$$

where C is a constant and m is the inverse slope exponent of the S–N curve.

The grid of the S–N curves in Eurocode, which is shown in Fig. 1, follows basically the recommendations of the International Institute of Welding (IIW) and of ECCS. Several existing codes had a spacing according to a few leading structural details. The associated S–N curves were named A, B, C, D et seq. The multiple individual structural details then have been sorted into one of those S–N curves (e.g. a root crack in fillet welds of a cruciform joint was an S–N curve named F). In IIW

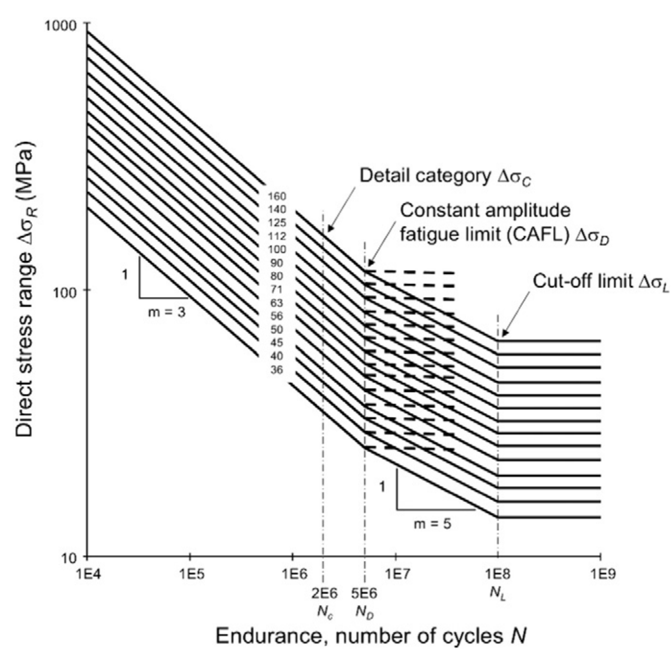

Fig. 1. Grid of Eurocode, based on IIW 1994.