Fatigue of road bridges

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Résistance à la fatigue des ponts-routes

Ermüdungsfestigkeit von Strassenbrücken

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SUMMARY

Fatigue load factors are derived from damage caused by fatigue and it is proposed that these factors should be described in terms of the number of load cycles for different details.

RÉSUMÉ

On montre de quelle façon les facteurs de charge de fatigue sont obtenus à partir des dommages en fatigue, et on propose que ces facteurs soient représentés en fonction du nombre de cycles de charge pour différents détails.

ZUSAMMENFASSUNG

Es wird gezeigt, wie aus Schadensfällen entsprechende Betriebslastfaktoren abgeleitet werden können. Ein Vorschlag wird gemacht, wie diese Faktoren für verschiedene Details je nach Anzahl der Lastwechsel gegliedert werden können.

1. PRELIMINARY REMARKS

Normally in road bridges a fatigue design is not necessary. But in the last years the volume of trafic grew very much and if at the same time the construction in view of fatigue is not well designed the fatigue problems come up in wind and torsional bracings, orthotropic plates (cross and longitudinal girders), transversal bracings and frames and expansion joints. If orthotropic plates are well designed there are no problems in fewer fatigues, but there are some problems today with the other secondary members. It is enjoyable that in the main structures of steel bridges no serious fatigue damages are known and damages in secondary members can be repaired relativly easy. From the known damages and damage analysis should be worked out systematically proposals for improving of the fatigue for this secondary members. In the following it will be tried to develope out of damages a simplified method for the fatigue design. At the present time because of lack of stress range spectras in the details because of lack of measurements the fatigue design according EC 3 [1] is very difficult.

2. STATE OF ART OF FATIGUE CODES IN AUSTRIA

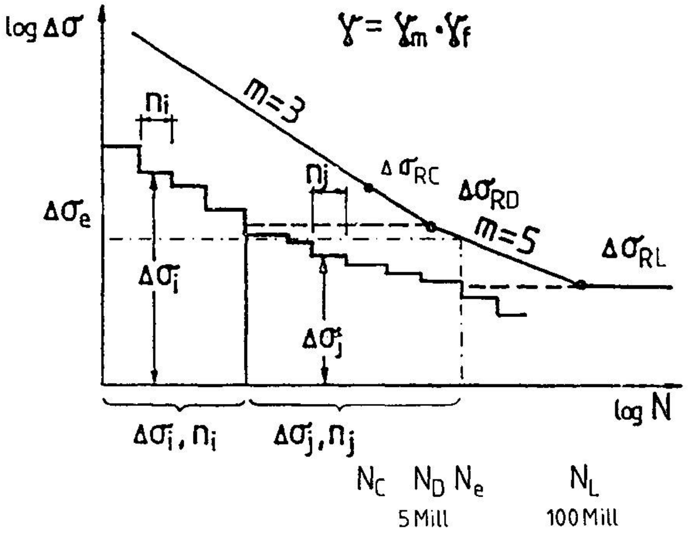
In Austria at the time the fatigue design is regulated in Ö-Norm B 4600 part 3. The bases is the constant amplitude fatigue limit. In the meantime in Austria it was developed a new "gide line for fatigue design" [2] which follows the recommendations EKS-TC 6 and Eurocode 3. Parallel Austria is working on Ö-Norm B 4300 part 5, which also follows the european concept. With the gide line according [2] it is possible to use the european concept before the new Ö-Norm is ready. But the difficulty is, that if you use this concepts on the load side, realistic fatigue loads have to be known and here is a big lack. Using deterministic loads for fatigue designs is not allowed and in the road norms no load models are fixed for the fatigue loadings. In the Ö-Norm there is only the advice that the details should be constructive designed in a way that fatigue problems don't occure.

3. FATIGUE DESIGN FOR ROAD BRIDGES, ACCORDING EC 3

The EC 3 gives now a bases for fatigue design with variable amplitude design limit, but the results of such calculation are not controlled by measurements. It will be shown now which difficulties in applications of EC 3 concepts in road bridges occure, if the fatigue design for secondary members has to be done. In comparison to railway bridges the fatigue design of road bridges is more complicated because the trace is changing and so the affect of action is non linear to the action. Also much less measurements have to be done to road bridges in comparison to railway bridges. If you start the fatigue design according Eurocode, first the fatigue loading has to be settled, than the volume of the trafic (mean-daily-number of lorrys) and the design life has to be fixed. With a load model some loading events, for example pulk of lorrys or override-manoeuvres have to be defined. With this loading events in each detail a design stress spectrum has to be calculated which depends non linear from the action. Normally the stress spectrum is calculated for the hot-spot-point and it is to be decided which point is the hot-spot. For the stress spectrum also changing of traces,distance of lorrys and imperfections of the

$$\Delta\sigma_e = \Big[\sum_i \Delta\sigma_i^3 \frac{n_i}{N_e} + \Big(\frac{\gamma}{\Delta\sigma_{RD}}\Big)^2 \cdot \sum_j \Delta\sigma_j^5 \frac{n_j}{N_e}\Big]^{1/3} \quad \text{(I)}$$

$$\Delta\sigma_e = \Big[\Big(\frac{\Delta\sigma_{RD}}{\gamma}\Big)^2 \cdot \sum_i \Delta\sigma_i^3 \frac{n_i}{N_e} + \sum_j \Delta\sigma_j^5 \frac{n_j}{N_e}\Big]^{1/5} \quad \text{(II)}$$



(I) $\Delta\sigma_e \geq \dfrac{\Delta\sigma_{RD}}{\gamma}$

(II) $\Delta\sigma_e < \dfrac{\Delta\sigma_{RD}}{\gamma}$

Fig. 1 Calculation of $\Delta\sigma e$

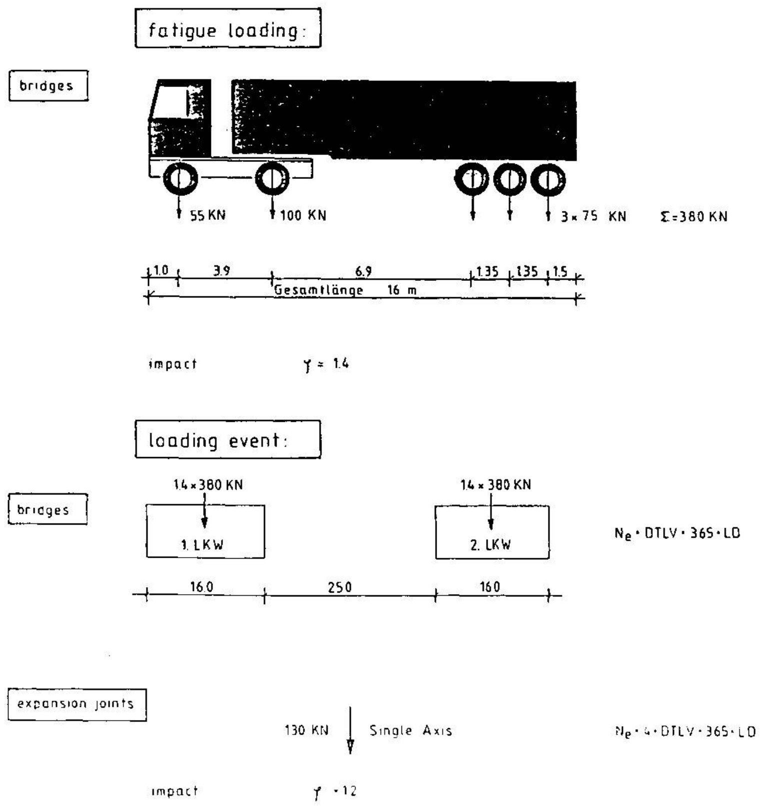


Fig. 2 Fatigue loading for bridges and expansion joints

road surface have to be fixed. In expansion joints the influence of the changing of the deflection, the dynamic stiffnes of plastics and the difference of phase between vertical and horizontal forces have to be considered. With the worst loading conditions the maximum stresses and stress spectras the hot-spot-point have to be calculated. The next step is to decide the detail categorie using the Miner-summation to get equivalent constant amplitude stress range. According to the assumptions the result of the calculation, for example life time vary in a wide range, because small changing in stresses give a big change of life time. This is due to the logarithmic form of the fatigue strength curve. So the calculation gives only tendences. Fig. 1 shows the fatigue sessment according EC 3. Computed stress spectras can not be controlled, they should be controlled by measurement. And here is a big lack of measurements.

4. CALCULATION OF FATIGUE LOADING FACTORS FROM DAMAGES

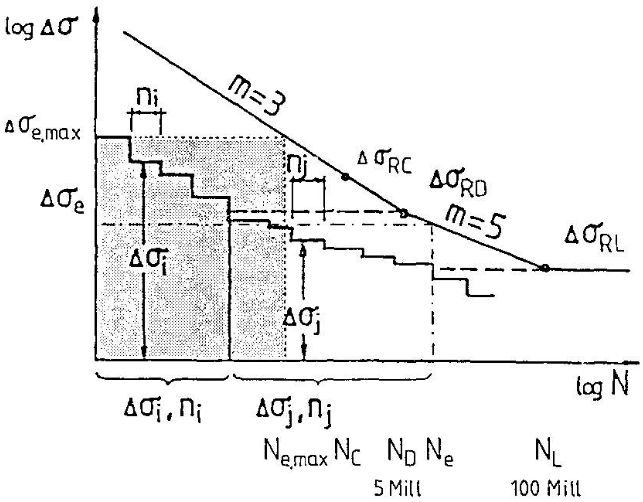
In the following it will be shown how by systematic treatment of damage analysis fatigue load factors of road bridges can be gained. First a realistic fatigue loading has to be set up and the mean-daily-number of lorrys has to be knewn. In bridges the fatigue loading which is realistic is a lorry with 380 KN and on the expansion joints a single axis of 130 KN is the maximum possible loading for fatigues (Fig. 2). In the hot-spot-point where the damage occured the maximum stress variation has to be calculated with this models. With the fatigue test the fatigue strength curve for the hot-spot-point has to be determined. Because now the life time and the maximum stress variation is known. The number Ne,max of load cycles to damage with constant maximum amplitudes can be gained (Fig. 3). In this number all non linear influences are included out of observations of the damage and with this number it is possible to calculate a realistic fatigue load factor. The fatigue load factor can be calculated according equation (1):

$$\alpha = \frac{\Delta\sigma_e}{\Delta\sigma_{max}} = \left(\frac{N_{e,max}}{5\cdot 10^6}\right)^{\frac{1}{3}} \cdot \left(\frac{5\cdot 10^6}{N_e}\right)^{\frac{1}{5}} \qquad (1)$$

where Ne the number of all load cycles during the life time is, This equation one get from the one step collective $\Delta\sigma_{max}$ and Ne, max in comparison to the one step collective $\Delta\sigma_e$ and Ne according EC 3. With the observations according Fig. 4 and 5 [3,4] using the proposed method you get for a torsional bracing α = 0,63 and for an expansion joint α = 0,34 (Fig. 6). If the results are drawn in relation to the parameter Ne / Ne,max you get Fig. 7. It shows that with growing number of load cycles the fatigue load factor is going down because of the fact that the propability of reaching the maximum stress variation goes down. Out of this it is proposed to give for different details different fatigue load factors, for example:

- details with low number of load cycles as torsion bracings and wind bracings, if they are acting in the main system
- details with a middle number of load cycles as orthotropic plates, cross bracings and frames
- details with high load cycle numbers as expansion joints.

The fatigue load factors would be accordingly 0,65, 0,45 respectively 0,35.



		N_L = 100 Mill
LD = 60 Jahre	DTLV 4000	N_e 88 Mill
	2000	44 Mill
	1000	22 Mill
		N_D = 5 Mill

$$\alpha = \frac{\Delta\sigma_e}{\Delta\sigma_{e,max}} = \left(\frac{N_{e,max}}{5\cdot 10^6}\right)^{1/3} \cdot \left(\frac{5\cdot 10^6}{N_e}\right)^{1/5}$$

Fig. 3 Determination of Ne,max

bracing:

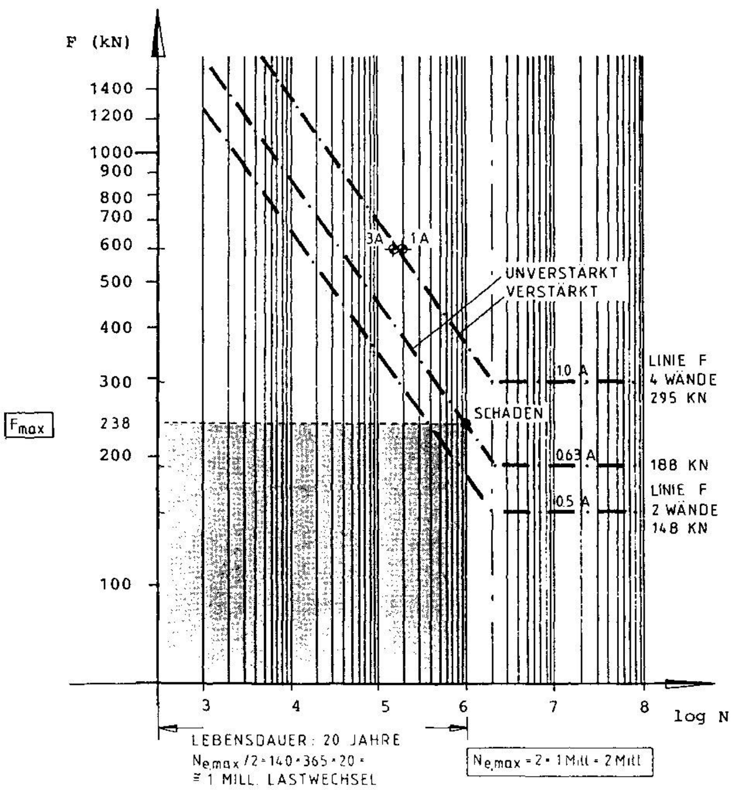


Fig. 4 Fatigue strength curve bracing

expansion joint :

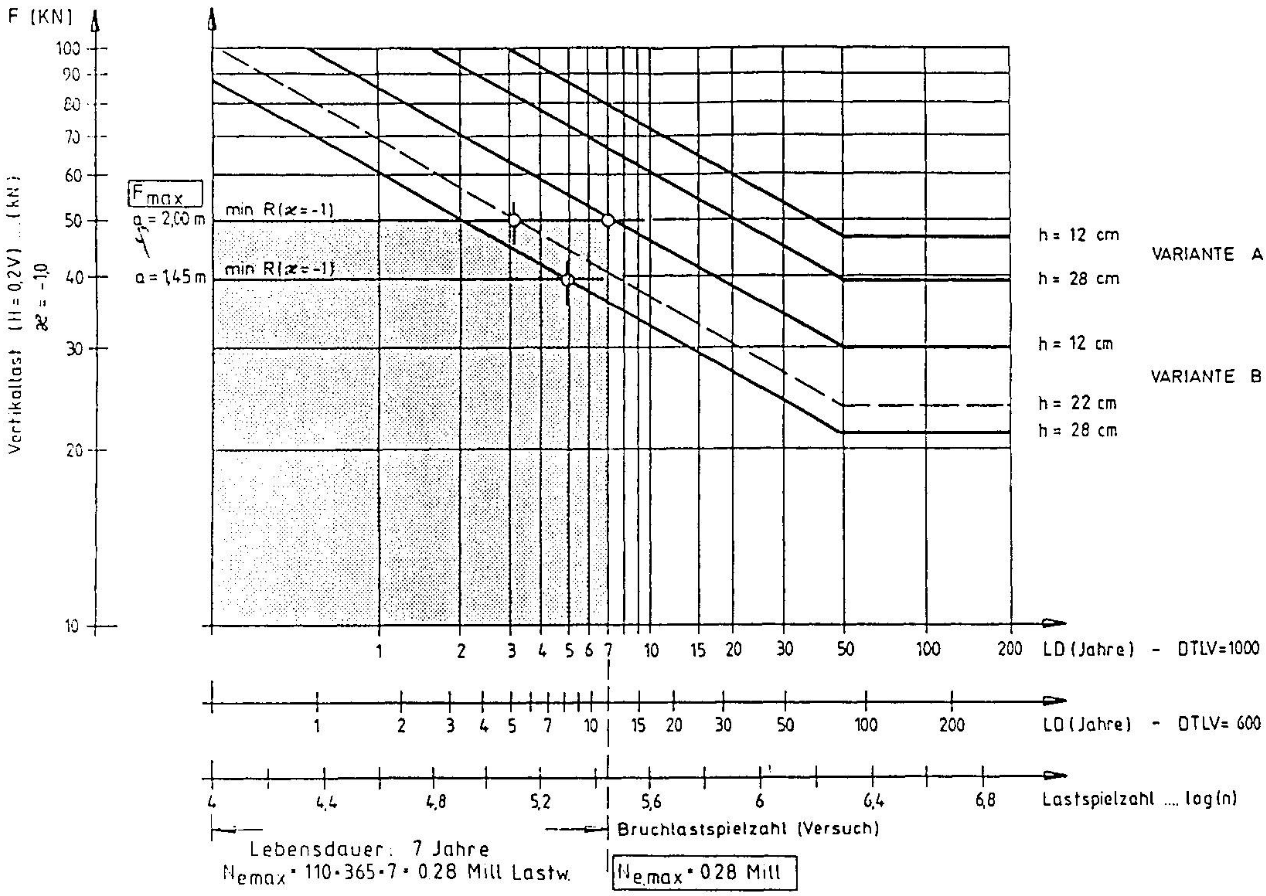


Fig. 5 Fatigue strength curve expansion joint

Lit.	[4]	[3]
member	bracing	expansion joint
LD Jahre	20	7
DTLV	1500	1000
N_e	1500 x 365 x 20 = 11 Mill	4 x 1000 x 365 x 7 = 10 Mill
$N_{e,max}$	20 x 2 x 140 x 365 = 2 Mill	7 x 110 x 365 = 0,28 Mill
α	0,63	0,34

Fig. 6 Determination of fatigue load factor α

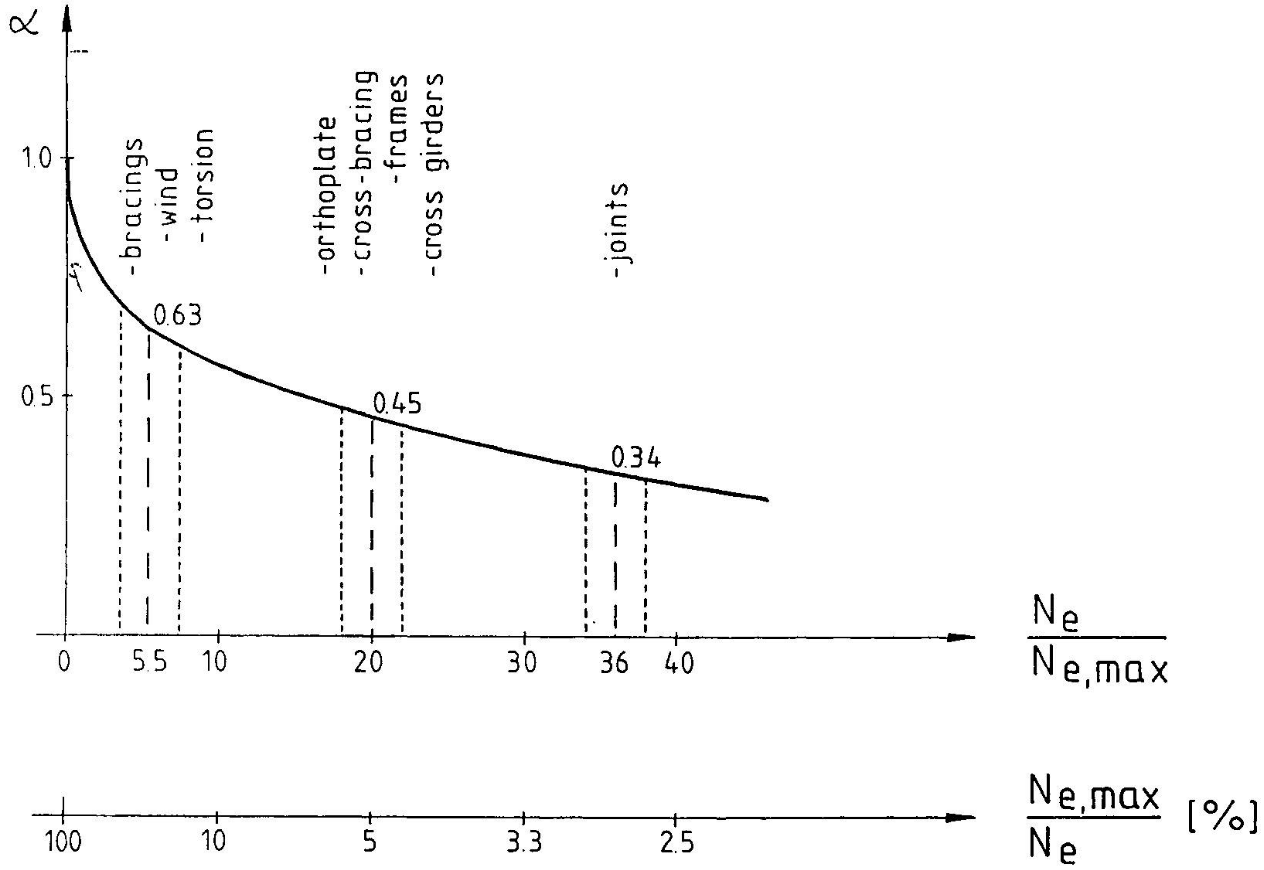


Fig. 7 Fatigue load factor for different details

Further research is necessary to verify this numbers. In any way it will be very helpful to give such fatigue load factors in EC 3 for different details to come to much easier fatigue assessment for secondary members of steel road bridges. Advantage is, that we are used to calculate maximum stresses and the fatigue tests have to be done at maximum stress variation. Further with the known fatigue load factors the remaining life time calculation is much easier and also results of computer simulation can be checked by this method.

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