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Evaluation of Existing Bridges under Actual Traffic

Évaluation des ponts existants sollicités par un trafic réel

Beurteilung bestehender Brücken unter wirklichem Verkehr

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SUMMARY

This paper presents a method for the evaluation of the reliability of existing bridges under the highest traffic loads and a method for the fatigue life assessment under repeated loads. A bridge classification is proposed for management. The fatigue life is assessed by a very simple calculation, taking into account the stress ranges produced by the fatigue load models defined in Eurocode 1-3 and the actual traffic on the bridge. The proposed method is also usable in the design of new bridges. An example illustrates the proposals.

RÉSUMÉ

Cet article présente une procédure d'évaluation de la sécurité des ponts existants sous l'action des charges maximales du trafic et une procédure d'évaluation de la durée de vie sous les charges répétées. Un classement des ponts est proposé pour la gestion. La durée de vie est estimée par un calcul simple qui tient compte des étendues de contrainte produites par les modèles de fatigue de l'Eurocode 1-3 et le trafic circulant réellement sur le pont. La méthode proposée est également utilisable pour l'étude de nouveaux projets de ponts. Un exemple illustre les propositions.

ZUSAMMENFASSUNG

Dieser Bericht stellt ein Verfahren für die Bestimmung der Zuverlässigkeit bestehender Brücken unter maximalen Verkehrslasten, und ein Verfahren für die Berechnung der Lebenszeit unter Verkehrslasten vor. Eine Brückenklassifizierung für die Verwaltung wird vorgeschlagen. Die Lebensdauer wird durch eine einfache Berechnung bestimmt, die die Spannungsschwingungen, erzeugt durch Ermüdungslastmodelle gemäss Eurocode 1-3 und dem wirklichen Verkehr, berücksichtigt. Der Vorschlag ist auch für die Bemessung neuer Brücken brauchbar. Ein Beispiel zeigt diesen Vorschlag.

1. INTRODUCTION.

Since 40 years, the road traffic has known a quick development, particularly in Europe. The development consists in the increase of the number of lorries, in the weight of the lorries, in the advent of tandem and tridem axles, and more recently in the increase of the percentage of loaded vehicles. The existing bridges have been designed taking into account load models corresponding, in the best situation, to the loads of lorries allowed at that time. The question of the reliability and the durability of the existing bridges arises for all bridges, but mainly if the gap between the loads of the models and the actual traffic is high. It is the case in Belgium [1]. During the development of the Eurocode 1-3 - Traffic loads on bridges - a lot of traffic loads have been recorded and used in order to define scientifically the characteristic loads and the fatigue loads [2]. Any existing Belgian bridge should satisfy usual safety factors under the loads defined in the Eurocode.

The aim of this paper is to show how it is possible to verify the ultimate limit state resistance and to estimate the fatigue life of critical details, taking into account the actual traffic on the bridge. The conclusions of such a study should either set forward the details that need particular attention during the bridge inspection because their fatigue life is short, or if necessary, define a limit of the loads allowed on the bridge.

2. DESIGN LOADS.

Since 1952 to 1993, the Belgian bridges have been designed considering the traffic loads defined in the code NBN 5, where in each lane a five axles vehicle of 320 kN (120 + 2 x 60 + 2 x 40) and a distributed load of 4 kN/m² were foreseen, these loads being multiplied by a dynamic factor never higher than 1,25 [1]. Figure 1 compares the total load Q located on a lane 3,5 meters wide and L meters long corresponding to NBN 5, to the loads given by the vehicles allowed to run in Belgium CR, to the actual vehicle loads running on European highways with a return period of one day Q_f (frequent load), and a return period of 1000 years Q_k (characteristic load) [3]. The figure shows that code loads are lower than the allowed loads, and sometimes close to $Q_f/2$ and $Q_f/3$.

The comparison of the loads located on a two lanes road on a L meters length, shows that the Eurocode loads, comprising in the first lane 2 axles of 300 kN spaced by 1,2 m. and a distributed load of 27 kN/m. and in the 2^d lane 2 axles of 200 kN spaced by 1,2 m. and of a distributed load of 2,5 kN/m², are always higher than the loads produced by a jam including 10 % of lorries.

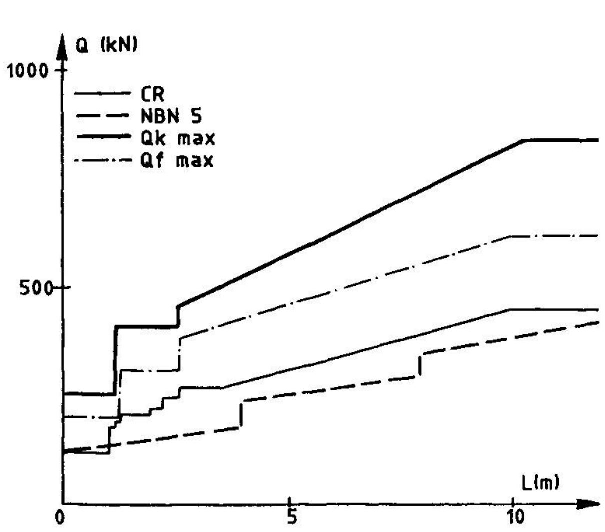

Figure 1 : Local loads.

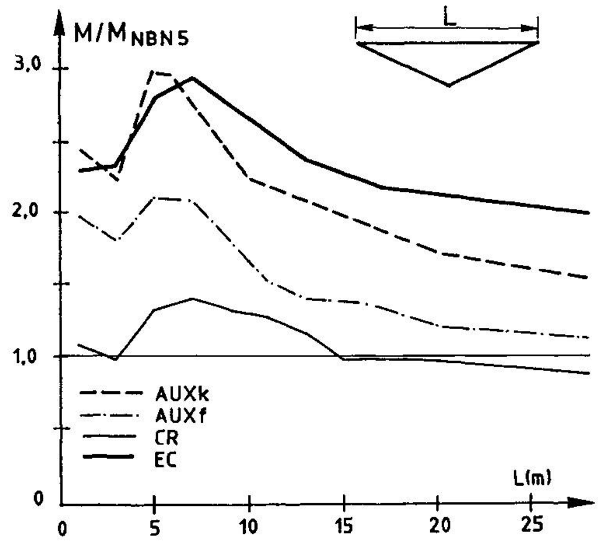

Figure 2 : Bending moment at mid-span.

Figure 2 compares the bending moment, dynamic effect included, obtained under the different loads at midspan of a simply supported beams supporting one traffic lane. The following conclusions are also valid for a lot of other influence lines [4]. The ratio of the load effect obtained by a load model and the NBN 5 loads are between, 1 and 1,4 for the allowed loads CR if L < 20 m., between 1,4 and 2,1 for the frequent loads of a traffic Auxf if L < 16 m. and between 2 and 3 for the characteristic loads of a traffic Auxk or of the Eurocode EC, if L < 25 m.

3. BELGIAN BRIDGES CLASSIFICATION.

Existing bridges have not been designed for the Eurocode loads. Nevertheless, failure produced by traffic loads are very rare up to now because a high safety factor is included in the design, so that the actual reliability of the bridges is comparable in Belgium and in the other European countries [5]. The Belgian National Application Document of EC 1-3 is now drafted to define 4 classes useful for the bridges managers :

class 1 concerns the design of new bridges, where the Eurocode loads are considered ;
class 2 concerns the repair of bridges, where the infrequent loads of Eurocode are considered (return period of 1 year : $\alpha_Q = \alpha_q = 0{,}8$) ;
class 3 concerns all existing bridges that are acceptable, where the frequent loads of Eurocode are considered (return period of 1 week : $\alpha_Q = 0{,}75$; $\alpha_q = 0{,}4$) and the safety factor γ_G and γ_Q are reduced ;
class 4 concerns the bridges where the vehicle weight allowed to run on the bridge is limited.

Table 1 : Bridge classes - N.A.D.

	1 european	2 belgian	3 acceptable	4 limited
$\alpha_{Q1} = \alpha_{Q2}$	1	0,8	0,75	0,75 C_R
α_{Q3}	0	0	0	0
α_{q1}	1	0,8	0,4	0,4 C_R
$\alpha_{q2} = \alpha_{q1} = \alpha_{qr}$	1	1	0,4	0,4 C_R
γ_G	1,35	1,35	1,1	1,1
γ_Q	1,35	1,35	1,2	1,2

A statical analysis has shown that the failure probability of a bridge of class 3 under the highway traffic is equal to 3.10^{-5} [3], value which is close to the one recommended by ISO [6]. For load effects influenced by not more than one vehicle, the frequent load should correspond to the vehicles defined in the Eurocode for the fatigue assessment FLM2. The load limitation in ciass 4 distinguishes between the load effects influenced by one or more than one vehicle, located in the same lane (long influence lines) or in two lanes (large influence surfaces).

4. FATIGUE LIFE ASSESSMENT.

4.1. Introduction.

The reliability of bridges satisfying class 3 is low, but acceptable. For the management of the bridges it is necessary to know the durability under the actual traffic flow. Eurocode 1-3 defines 5 fatigue load models [2]. FLM1 and FLM2 define frequent loads ; if the stress range produced by this load is below the fatigue limit no fatigue damage is expected. FLM3 and FLM4 define equivalent loads or mean loads, usable for the fatigue assessment. FLM1 is derived from the characteristic load model : $Q_f = 0{,}7\ Q_k$ for the axle loads and $q_f = 0{,}3\ q_k$ for the distributed loads. FLM2 and FLM4 define a set of 5 lorries by geometry, axle loads and frequencies, and are usable for effects produced by one lorry alone. FLM3 comprises a single symmetrical vehicle with 4 axles of 120 kN ; but we have demonstrated elsewhere the need of the second vehicle for long spans with only 30 % load and located 40 m. behind the first vehicle [3]. FLM5 considers a whole load spectrum and is used only for special cases. The fatigue life should be calculated for each detail of an existing bridge knowing

the influence area and the traffic flow. The total number of lorries crossing on the bridge should be estimated by visual counting. This counting should give the frequency of each type of vehicle. Taking into account the loads defined for FLM3 or FLM4 in EC 1-3, a fatigue assessment is possible. A more complete information of the vehicle loads requires records by a weight device, but this is only necessary if the lorry loads are very different of the loads given in the Eurocode, which correspond to a heavy long distance traffic in Europe.
The method proposed for the assessment of the fatigue life is described here for existing bridges, but it is also applicable for the fatigue verification needed in the design of new bridges. As the method considers fatigue resistance defined by SN curves, it concerns steel as well as concrete elements.

4.2. Fatigue life assessment.

The fatigue life assessment is carried out in two steps. The first step should conclude whether fatigue damages are expected or not. If fatigue damages are expected, the fatigue life is calculated.

1° Fatigue life is unlimited if $\Delta\sigma_{M1} \leq \Delta\sigma_D / \gamma_{Mf}$, where,

$\Delta\sigma_{M1}$ is the highest stress range produced by FLM1,

$\Delta\sigma_D$ is the fatigue limit under constant amplitude,

γ_{Mf} is the partial safety factor.

For short spans, i.e., for spans shorter than 30 m. if the influence line comprises areas alternatively positive and negative or for the addition of the two contiguous spans shorter than 30 m. if the sign of the areas are the same, FLM1 may always be replaced by FLM2.

2° Fatigue life is calculated.

$$\Delta\sigma_{M3} \leq \left(\frac{N_D}{N_t}\right)^{1/m} \frac{1}{C_t} \cdot \frac{\Delta\sigma_D}{\gamma_{Mf}} \text{, where,}$$

$\Delta\sigma_{M3}$ is the highest stress range produced by the vehicle of FLM3 defined in EC 1-3 completed by a 2d vehicle located 40 m. behind [3],

N_D and $\Delta\sigma_D$ corresponds to the fatigue limit on the SN curve ($N_D = 5.10^6$ in EC3 [7]),

$$C_t = \left(\frac{\Sigma n_{ij} \cdot Q_i^m}{\Sigma n_{ij}}\right)^{1/m} \left[\frac{\Sigma n_{i1} \cdot Q_i^m}{\Sigma n_{i1}}\right]^{-1/m}$$

is a factor to consider for traffic with an other composition as the long distance traffic,

Q_i is the equivalent weight of vehicle i if the stress range is produced by all axles of each vehicle ; if each axle produces a stress range, each axle weight is considered successively,

n_{ij} is the frequency of vehicle in traffic j,

n_{i1} is the frequency of vehicle i in long distance traffic,

m defines the slope of the SN curve (m = 5 in EC 3 and m = 5 to 9 in EC2 [7] [8]).

N_t is the number of cycles produced by the traffic.

The value of N_t is different if one or more lanes influence the stress range :

a) If the stress range is produced by the traffic running in two slow lanes of the road and if the ratio $\Delta\sigma_{M3/2}/\Delta\sigma_{M3/1}$ between the effects of each lane is higher than 0,5, the vehicle of FLM3 is running successively in each slow lane, and

$$N_t = N_{t1}\left[1+\left(\frac{\Delta\sigma_{M3/2}}{\Delta\sigma_{M3/1}}\right)^m\right]$$

$N_{t1} = N_a \cdot N_{obs} \cdot k_1 \cdot k_2$, where

N_a is the life time in years,

N_{obs} is the number of lorries per year running on one slow lane,

k_1 is the ratio of the number of loaded lorries to the total number of lorries ; for heavy traffic $k_1 = 2/3$

$$k_2 = 0,6 + \frac{1}{0,25L} \quad \text{if} \quad 1,18 \text{ m.} < L < 10 \text{ m., L the span length in meters,}$$

$$k_2 = 4 \quad \text{if} \quad L \le 1,18 \text{ m.}$$
$$k_2 = 1 \quad \text{if} \quad L \ge 10 \text{ m.}$$

b) If condition a) is not satisfied, only the traffic running on one lane is considered, and the traffic running in fast lanes may be neglected because it represents not more than 10 % of the traffic flow of the slow lane, $N_t = N_{t1} = N_a \cdot N_{obs} \cdot k_1 \cdot k_2$

In all cases, the life time becomes $N_a = \dfrac{N_{t1}}{N_{obs} \cdot k_1 \cdot k_2}$

If, for local effects, the stress range depends highly on the geometry of the vehicles, the vehicle of FLM3 is replaced successively by each vehicle of FLM4 with the appropriate frequency in order to obtain the equivalent stress range.

5. EXAMPLE.

Figure 3 shows the cross section of a common highway bridge in Belgium.

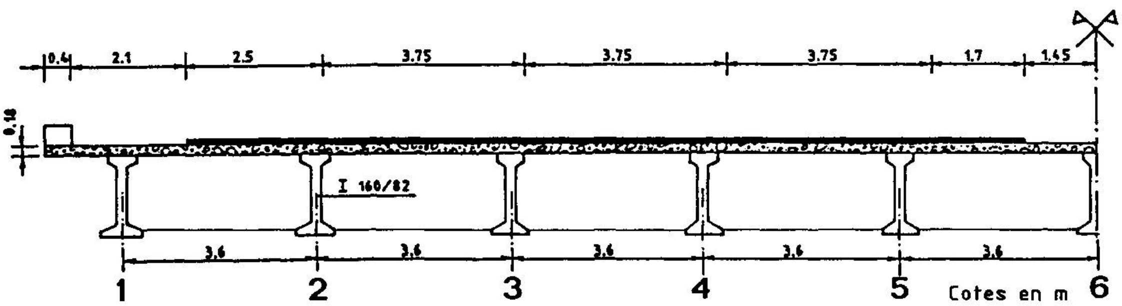

Figure 3 : Cross section of a Belgian bridge.

Table 2gives for three sections,

M_G	moment under dead load,
M_Q	moment under load model of NBN 5,
M_R	ultimate resistance moment,
M_{Qk}	moment under characteristic load of EC 3-1,
$\Delta\sigma_{M3}$	stress range in the reinforcement corresponding to FLM3 of EC 3-1,
N_{veh}	number of lorries before fatigue crack.

The main reinforcement in the span satisfies class 2, the section on the support satisfies class 3, but transverse bars in span do not satisfy class 3and a reduction of the loads of the vehicles allowed to cross the bridge is required following class 4.

A more accurate analysis using FLM4 shows that 85 % of the total damage produced by a long distance traffic results from the 2^d axle of half trailer vehicles.

The fatigue life for main bending is longer than 100 years for highway traffic (40.000 lorries per week) but for transverse bending the fatigue life is very short : 4 years for main road traffic of category 3 (2.500 lorries per week). The durability of such a bridge depends on the contribution of the pavement with the concrete slab in a composite effect : each reduction of the stress range by 8 % doubles the fatigue life.

Table 2 - Moment in kN m/m.

	Section on support 2	Section in span	
		M_x	M_y
M_G	- 7,16	3,40	0
M_Q	- 47,04	31,36	-
$M_R = 1,5\ (M_G + M_Q)$	- 81,3	52,14	10,42
M_{Qk}	- 75,11	40,23	17,60
M_{Qf}	- 55,81	27,11	13,20
1) $1,35\ (M_G + M_{Qk})$	- 111,1	58,9	23,76
2) $1,35\ (M_G + 0,8\ M_{Qk})$	- 90,8	**48,0**	19,00
3) $1,1\ (M_G + 1,2\ M_{Qk})$	**- 70,7**	36,3	15,84
$\Delta\sigma_{M3}$ (N/mm²)	105	87	190
$N_{véh}$ (10^6)	170	1080	0,54

6. CONCLUSIONS.

The proposed method allows a classification of the existing bridges regarding their ultimate limite state. The very simple method for the fatigue assessment of existing bridges proposed is also usable for the fatigue verification in the design of new bridges. The example has shown the difficulties of an actual assessment of existing bridges, where the durability depends of the beneficial effect of the asphalt surfacing.

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