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ENV 1991 - Part 3 : Traffic loads on bridges
Calibration of road load models for road bridges

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Summary

Part 3 of Eurocode 1 defines the traffic load models to be used for the design of bridges. The load models representing road traffic loads have been calibrated on traffics recorded in Europe in the eighties. This paper shows how the representative values of these loads have been determined.

1. Introduction

The present paper concerns background studies about the calibration of the load models representing the actions induced by the road traffic [1]. Its content allows to outline, together with the topics discussed in [2], a complete description of the studies carried out in the definition of actions on road bridges. The models have been defined so that it is possible to obtain correct bridge design, following the requirements of the design codes, mainly EC 2-2 Concrete Bridges, EC 3-2 Steel Bridges, EC 4-2 Composite Bridges.
The aim of the calibration is to obtain load models which are able to reproduce as well as possible the effects induced by the road traffic, being at the same time very simple and easy to use. In order to do this, it has been necessary first of all to evaluate the so called «target values», representing the real traffic effects.
Taking into account the needs concerning ultimate and serviceability limit states checks as well as fatigue assessments, target values have been defined for a lot of load effects, regarding various influence lines and bridges spans, considering several traffic scenarios, several extrapolation methods and dynamic effects induced by different roughness of the pavements. In this paper, a wide set of comparisons between the target values and the EC 1-3 load model values is also reported for each case.

2. Extrapolation methods

The choice of the main load model and its calibration require preliminarly the knowledge of the effects induced by the real traffic on the bridge, in terms of their characteristic, infrequent and frequent values, which must be reproduced by the load model itself.
The procedure to be followed to evaluate these target values is not obvious. In fact, because the recorded traffic data concern flowing traffic on time intervals limited to few hours or to few days, it is necessary to study how to transfer these data to the whole life of the bridge, taking also into account the extreme traffic situations which can happen on one or on several lanes.
In a very general scheme, the procedure can be summarised as follows : the most representative traffic samples are considered to cross the bridge, in such a way that the histograms of the extreme values of the considered effects are determined, and subsequently, using a suitable extrapolation method, the values with prefixed return periods are evaluated.
Traffic samples, traffic situations, hazard scenarios, as well as set of influence lines considered in the calibration, are outlined in [2].
To evaluate both, the extreme values of axle and lorry loads and the extreme values of the traffic effects, basically three different extrapolation methods have been adopted, using respectively, the half-normal distribution, the Gumbel distribution and the Montecarlo simulation [3], which are shortly described in the following.

2.1. Half-normal distribution

The method is based on the hypothesis that the queue of the extreme values distribution of the stochastic variable x is gaussian, so that the upper part, for $x \geq x_o$, of the histogram of the effect induced by the real traffic can be fitted with a gaussian curve through a suitable choice of the parameters of the curve itself. Generally, the parameter x_o is close to the last mode of the histogram [8][9].
The value x_R, corresponding to the return period R, is given by $x_R = x_o + \sigma.Z_R$, being Z_R the upper α-fractile of the standardised normal variable $Z = (x-m)/\sigma$. In the present case $\alpha = (2.N_T)^{-1}$, where N_T is the total number of events during the period R.

2.2. Gumbel distribution

Under hypotheses similar to those illustrated in the previous point, the extreme values distribution can be represented using the Gumbel distribution (or extreme value I type distribution), which is completely described by the parameters u, representing the mode of the distribution, and α', depending on the scattering of the distribution.
The parameters of the Gumbel distribution can be obtained, starting from the histogram of the extreme values, as $u = m - 0,45.\sigma$ and $\alpha' = (0.7797.\sigma)^{-1}$, where m and σ are, the mean and the standard deviation of the histogram. The value x_R is then given by $x_R = u + y.\alpha'$ being $y = -\ln[-\ln(1 - R^{-1})]$ the reduced variable of the distribution.

2.3. Montecarlo simulation

The Montecarlo simulation is based on the automatic generation of a set of extreme traffic situations, starting from the recorded traffic data, so that it is possible to obtain the extreme value sample on which the extrapolation method is applied.

The sample can be generated in several ways, depending essentially on the number of applications of the method itself.
The most intuitive procedure consists in the application of the method several times. The lorries crossing the bridges are chosen from a suitable garage, i.e. a set of standard vehicles representing the most common real lorry schemes. Lorry types, axle loads, interaxle distances as well as intervehicle distances are obtained applying repeatedly the Montecarlo method, on the basis of the statistical parameters derived from the analysis of the recorded data.
Beside that, an alternative procedure, more complex but very efficient, has been adopted : in this one the aim of the Montecarlo simulation is to obtain, using the parameters of the extreme values distribution obtained with the recorded traffic data, a statistical sample of the effects. In this way the application of the Montecarlo method is limited only to the final steps on the procedure, in order to determined the input data for the calculation of the parameters of the Gumbel type distribution [4].

3. Dynamic effects

Besides the extrapolated values, the determination of the target values requires the evaluation of the dynamic effects due to the interaction between the vehicles and the bridge.
In order to obtain the values of the dynamic load effects to be used for the calibration for serviceability limit states, for ultimate limit states as well as for fatigue assessments, special studies have been carried out by an ad hoc Working Group [5].

3.1. Inherent dynamic increment

Because the recorded traffic data have been obtained by measurements from flowing traffic, they contain already dynamic increments, so that it is necessary to correct them with the inherent impact factor. The inherent impact factors for recorded traffics have been determined by computer programmes in which measurements are simulated assuming rigid ground with good surface roughness, and vehicle loads are represented as sequence of static actions.
Regarding the extreme values of Auxerre data relevant for the ULS consideration, an inherent impact factor φ_{inh}= 1.10 has been found, while for loads belonging to the fractile ranges between 10 % and 90 %, relevant for SLS and fatigue, there is no significant difference between static and dynamic distribution, so that it results $\varphi_{inh} \cong 1.00$.

3.2. Impact factor

The dependence of the impact factors on the model parameters, like bridge type, static scheme, span, fundamental frequency, damping quality, dynamic characteristics of the vehicles, roadway roughness, vehicle speed and so on, has been preliminary investigated in order to determine the weight of each parameter.
Subsequently, using computer programmes, a lot of numerical simulations has been carried out for several bridge types and for various traffic scenarios, with medium or good roadway roughness, evaluating the corresponding global dynamic increments. Beside that the local dynamic effects as well as the timber effect due to a concentrated irregularity, 30 mm high and 500 mm wide, simulating uneven transition joint, lost board or ice slab, has been determined.

The results of each numerical simulation is a time history of the considered effect, from which the ratio between the extreme dynamic response and the extreme static response of the bridge can be determined. This ratio is commonly said physical impact factor $\varphi = \max_{dyn}/\max_{stat}$.
The above defined physical factor refers to a particular loading situation and depends on such a variety of parameters that cannot be directly employed for load model calibration. In fact φ is usually high for light vehicles and low for heavy vehicles, while the target values depend mainly on the extreme values of the dynamic distribution.
For code purposes, the dynamic magnification can be taken into account directly, referring to the distribution of the dynamic effects, or, in an alternative way, increasing the static distribution by an impact factor φ_{cal} ratio between dynamic and static values corresponding to the same x-fractile $\varphi_{cal} = E_{dyn(x\text{-fractile})}/E_{stat(x\text{-fractile})}$.
Of course, φ_{cal} is purely conventional because the static and dynamic x-fractiles don't correspond necessarily to the same load condition. The characteristic values of the conventional impact factors φ_{cal} have been determined, using Auxerre data for flowing traffic, simulating a lot of influence lines, span lengths and pavement roughnesses occurring in actual bridges. The results are summarised in Figure 1.
The dynamic target values of the effects can be then computed, starting from the effect E_{stat}, obtained using the recorded traffic data together a suitable extrapolation method, as $E_{dyn} = E_{stat} \cdot \varphi_{cal} \cdot \varphi_{local}/\varphi_{inh}$, where φ_{local} represents the impact factor for local effects.

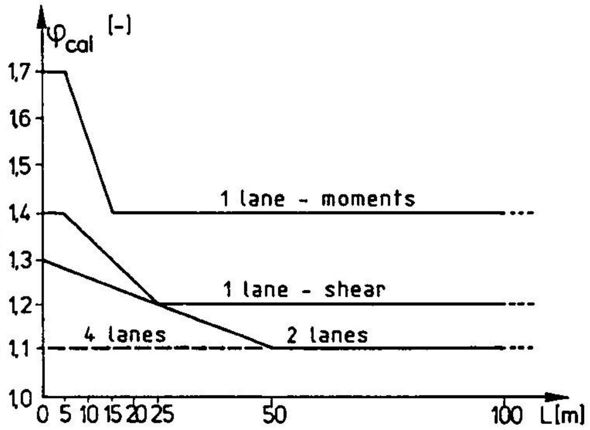


Fig. 1 Impact factors

3.3. Damage equivalent impact factor

The impact factor for fatigue design is defined as the ratio between the fatigue damage induced by the dynamic stress history and the fatigue damage induced by the static stress history.
This definition leads to a damage equivalent impact factor φ_{fat} expressed by

$$\varphi_{fat} = \left[\sum n_{i,dyn}\left(\Delta E_{i,dyn}\right)^m \Big/ \sum n_{i,stat}\cdot\left(\Delta E_{i,stat}\right)^m\right]^{1/m}$$

where ΔE_i are the effect ranges and m is the slope of the S-N curve.
This definition allows to obtain an increased histogram, leading to the same damage as the original dynamic histogram, simply multiplying all the stress amplitudes of the static histogram by the constant impact factor φ_{fat}.

4. Safety factors γ_Q and reduction factor ψ_1

4.1 General

The safety elements for actions $F_{d1} = \gamma_{F1} \cdot F_{k1}$ and $F_{d1} = \gamma_{Fi} \cdot \Psi_i \cdot F_{ki}$
can only be determined by considering both,

the action side	$S_d = S\,(\gamma_{F1} \cdot F_{k1}\,,\ \gamma_{Fi} \cdot \psi_i \cdot F_{ki}\,,\ a_{nom})$
and the resistance side	$R_D = R\,(f_k\,,\ a_{nom})/\gamma_M$
and the relevant limit states.	

4.2. Procedure and results for γ_Q

The following procedure has been adopted to determine the magnitude of the safety factor $\gamma_F = \gamma_Q$ to be applied to traffic loads [6] :

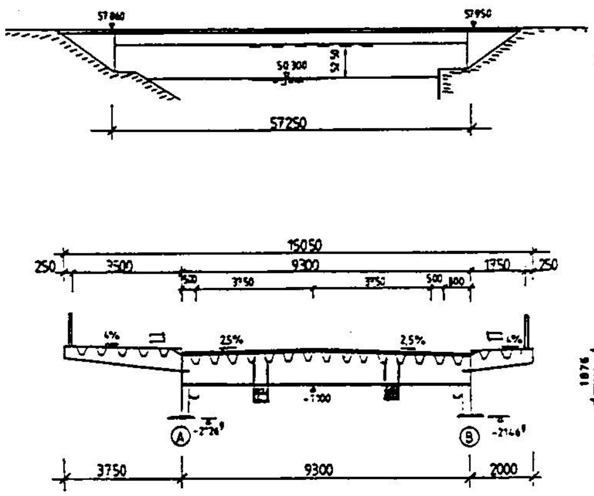


Fig. 2a. Single span bridge K 210

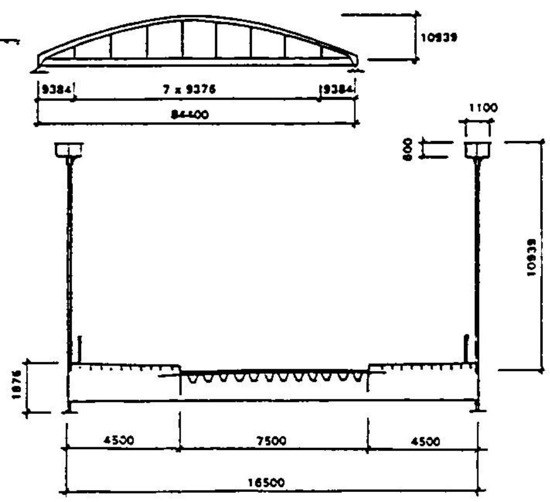


Fig. 2b. Tied arch bridge K 138

1. Two steel bridges were selected (Fig. 2a and 2b) for which the first order reliability method was used to determine the safety index β for various elements considering.
   - bridges loaded by the Auxerre-traffic used to define the main load-model,
   - the limit states constituted as follows :

     ULS : attainment of the first yielding ;

     SLS : attainment of a defection limit ;

     Fatigue : attainment of a required service life.
   - all actions in combination with the traffic loads (selfweight, wind, temperature gradient) and all bridge properties relevant for the limit states described by statistical data independent on the bridges selected for the calibrations.
2. From the reliability studies the β-values as indicated in Figure 3 were determined, from which the following requirements for target β-values to be applied to parameter studies were taken :

   β = 6,00 for ULS and

   β = 3,00 for SLS and fatigue.

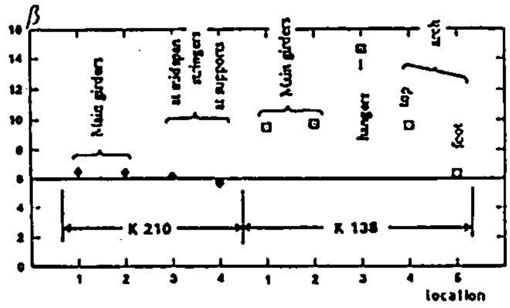
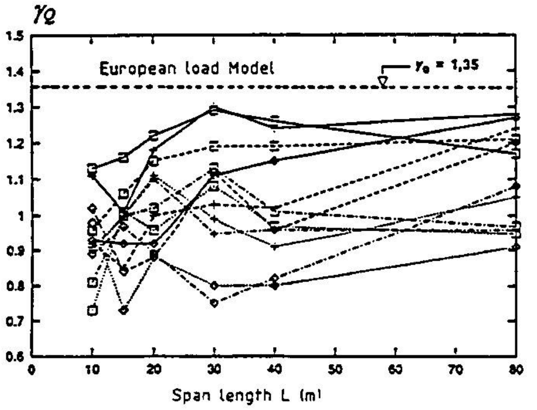


Fig. 3 : β values for bridges elements.

3. For a set of representative bridge systems (single spans or continuous spans, one or several lanes) a probabilistic design was carried out using the same statistical parameters as used for the calibration described in 1 and 2. The probabilistic design resulted in the required section moduli $W_{required}$.
4. The required safety factor γ_Q to be applied to the Eurocode traffic load model was then determined by comparing the design values M_{Qd} from the probabilistic design,
   $M_{Qd} = f_y . W_{erf}/\gamma_M - M_g.\gamma_G$
   where f_y and γ_M = 1,10 were taken from Eurocode 3 and
   M_g and γ_G = 1,35 were taken from Eurocode 1, with the action effect from the traffic load model LM,
   $M_{Qd} = \gamma_Q . M_Q^{LM}$
   This comparison implies that the combination rule is $\gamma_G .G + \gamma_Q. Q$.

Figure 4 gives the results of that comparison that yielded to the value γ_Q = **1,35** that was recommended to be applied to the European load model.



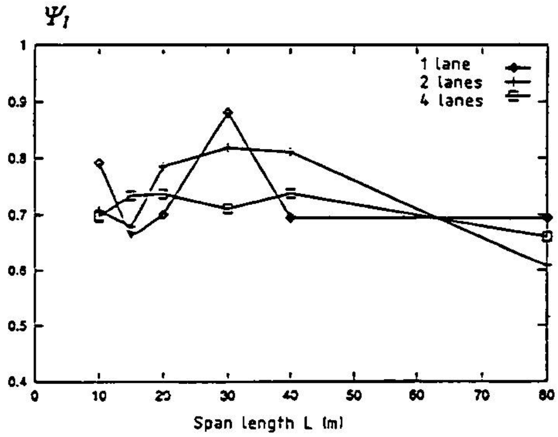


Fig. 4 : γ_Q values for bridge elements *Fig. 5 : Ψ_1 values for bridge elements*

4.3. Results for Ψ_1

For determining the reduction factor Ψ_1 for the serviceability limit state of deflection the same statistical parameters were used as for the parameter study for ultimate limit state design. Deflections are caused both by traffic loads and by temperature differences that were considered in combination.

The comparison was performed on the basis of the required second moment of area $I_{required}$ that was determined for the set of representative bridge systems by a probabilistic calculation on one side and by using the characteristic load models in Eurocode 1 with a reduction factor Ψ_1 on the other side. This comparison leads to

$$\psi_1 = M_{Qd\,serv} / M_{Qk}^{LM,\,ULS}$$

Figure 5 gives the results for the required ψ_1-values for single span bridges. The value adopted in EC 1-3 is ψ_1 = **0,75**.

4.4. Results for fatigue

A comparison of the required section moduli $W_{required}$ from the probabilistic fatigue calculation and from the fatigue loading model FLM 3 in EC 1-3 is given in Figure 6a for the main girder of a single span bridge with a span length of 20 m and in Figure 6b for a span length of 80 m. Apparently β = 3.0 is reached for smaller spans only, whereas the FLM-3 model must be modified for longer spans (see section 7.2)

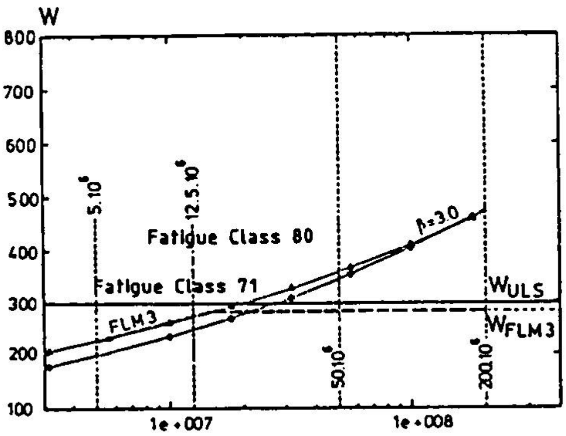


Fig. 6a. $W_{required}$ - Main girder, L = 20 m

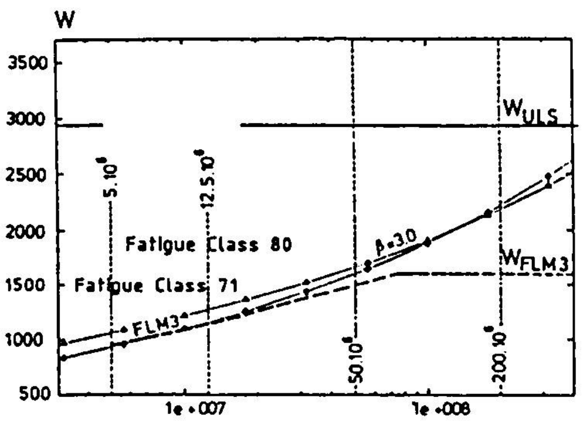


Fig. 6b. $W_{requied}$ - Main girder, L = 80 m

5. Characteristic loads

5.1. Procedure of calibration

The characteristic loads have been defined for a return period of 1000 years. The importance of the choice of the return period is shown in section 6.2.
The characteristic loads model have been defined considering several traffic scenarios and influence lines [2], with reference to the Auxerre traffic, recorded on the motorway Paris-Lyon in France. The calibration has shown that two influence lines are determining : M_o, the bending moment at mid span of a simple supported beam and M_2, the bending moment on the central support of a beam with two spans [7]. The results given later for these two lines are sufficient to illustrate the whole calibration studies.
The members of the Project Team have proposed several traffic scenarios and several extrapolation methods. All the proposed target values have been compared on graphs giving a fictitious load Q' in function of the span length L : Q' = k.M/L or Q' = k.V, where M is a bending moment,V is a shear force, and k is a factor depending on the type of load effect. On such a graph, a load effect produced by a constant load is represented by an horizontal straight line and a load effect produced by a constant uniform distributed load is represented by a slopping straight line.
The load effects produced by the load model should cover, as far as possible, all proposed target values, because all proposals have to be considered.
The development of the characteristic load models was carried out studying, first the general shape of the load model on lane 1, than the local loads on lane 1 and finally the load model on a carriage-way with several lanes.

5.2. General shape of the load model on lane 1

The target values, dynamic effect included, proposed by five members of the Project Team are reported on Figures similar as Figure 7 [7]. The Figures showed that for short spans (below 30 m. to 50 m.) free traffic produces higher moments than congested traffics for reason of the dynamic effect. The envelope of all results should be represented by a straight line, that will say that the load model producing the moments may be composed by a concentrated load and a constant uniform distributed load. Regarding all influence lines, the concentrated load is comprised between 450 to 720 kN, values close to the characteristic weight of a vehicle, and the distributed load is comprised between 21 to 28 kN/m, value close to the mean linear weight of the lorries running in jam. The curve LM1, given on Figure 7 corresponds to the load model 1 prescribed in the EC, where the local load is equal to 600 kN and the distributed load is equal to 27 kN/m. This model gives too high values for short spans and in some cases too low values for long spans. But, as for long spans a carriage-way comprises always more than on lane, this problem is to reconsider in section 5.4.

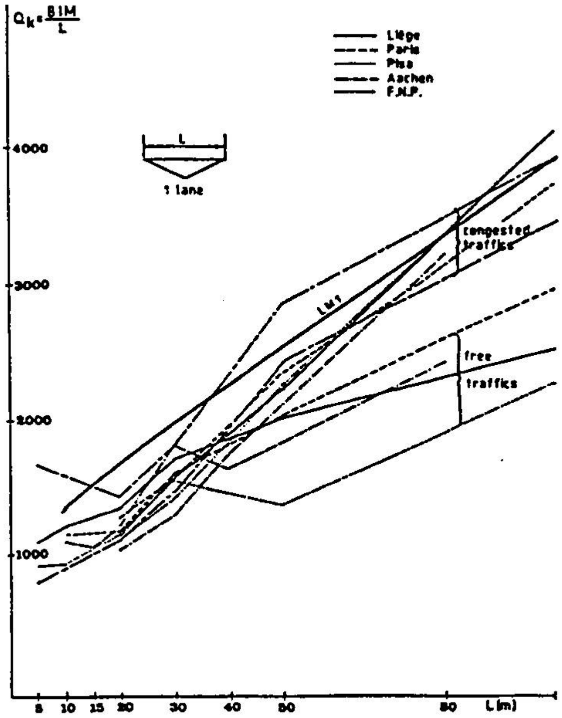


Fig. 7. Target values - M_o - 1 lane

5.3. Local loads on lane 1

The position and the weight of the axles of actual lorries are relevant for local load effects. The extrapolation of the recorded loads available leads to characteristic loads, without any dynamic effect ; that are given in the table.

	characteristic loads (kN)		
	Min.	Max.	Auxerre traffic
simple axle	17	250	230
tandem axle	280	400	360
tridem axle	350	480	480
lorry	560	870	750

The heaviest vehicle is shown on Figures 8a and 8b. In one case, on Figure 8b, a tandem axle of 2 x 200 kN of an other vehicle produces the highest effect.The Figures compare the target values, dynamic effect included, with the values produced by three load models comprising respectively, 1, 2 or 3 axles.
The Figures show that the model with one axle of 600 kN (LMa1) gives too high values for short spans and is therefore not appropriate for the calculation of local effects. The model with 2 axles of 300 kN (LMa2), that corresponds to model 1 of the Eurocode 1.3., gives the best fit, even if the moments at midspan are too high for spans above 7 m. (up to 18 %, see Figure 8a) and the moments on support are too low for spans comprised between 4 and 9 m. (up to 10 %, see Figure 8b).

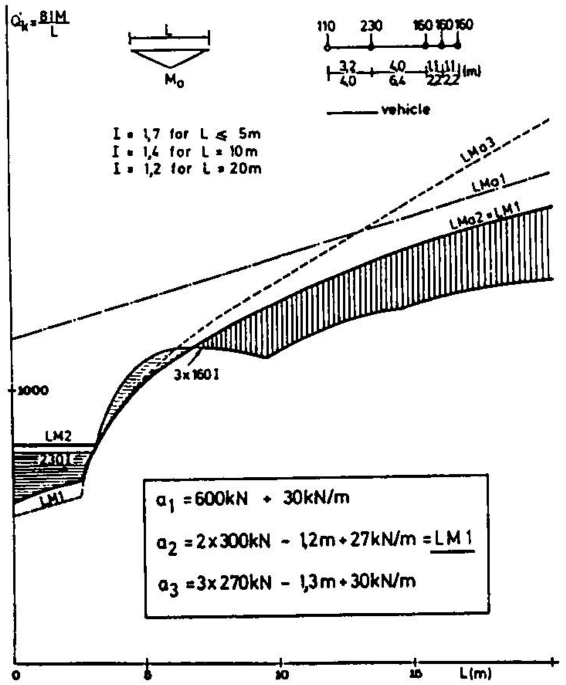


Fig. 8a . Target values - M_o - Local effects

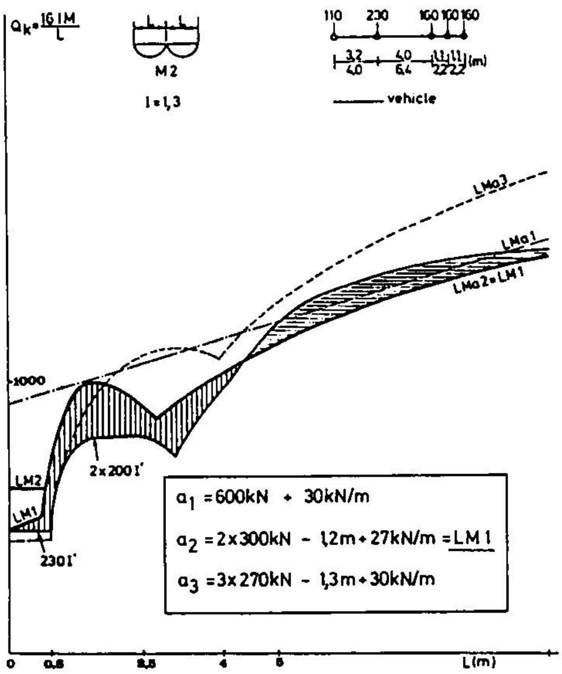


Fig. 8b. Target values - M_2 - Local effects

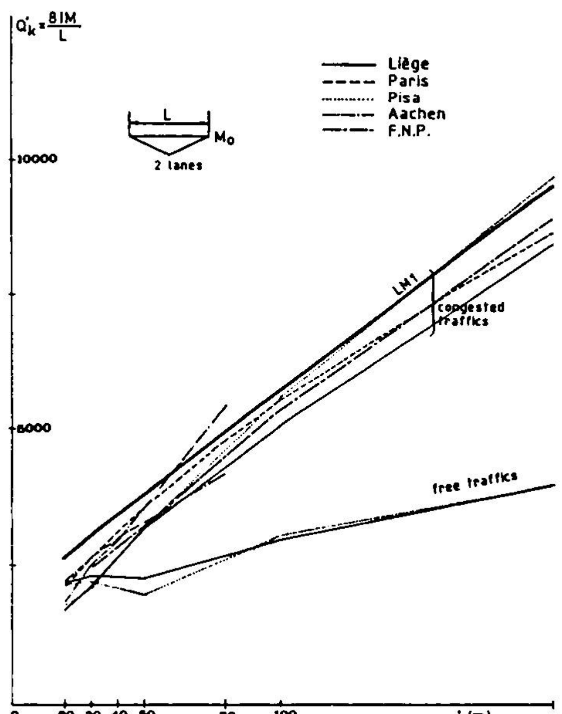


Fig. 9. Target values M_o - 2 lanes

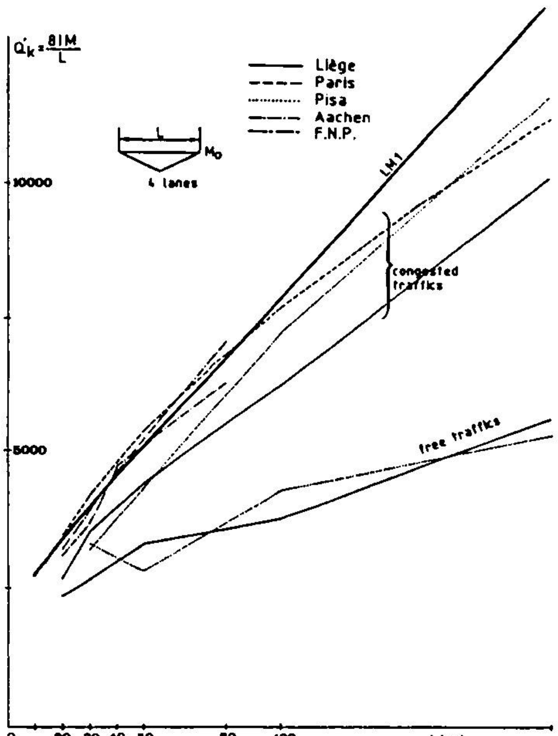


Fig. 10. Target values M_o - 4 lanes

As for short spans the model gives too low load effects, model 2, comprising one axle of 400 kN, has been introduced in the code. This single axle corresponds to the heaviest extrapolated axle load (250 kN), multiplied by a dynamic factor I = 1,6.

5.4. Load model on a carriage way with several lanes

Figure 9 shows that for a two lanes bridge, congested traffics are mainly to consider for the determination of the load model.
Figure 10 shows that for a four lanes bridge, LM1 is very close to the highest target values proposed, but the distributed load should be reduced for spans longer than 100 m., while in some cases the local load is too low.
The load model of EC 1-3 will cover all traffic scenarios envisaged if the distributed load on lane 2 (2,5 kN/m²) is increased and axle loads should be applied on each lane [7].
The distributed load could be reduced on large bridges, having four or more lanes and spans longer than 100 m, but 2,5 kN/m² seems an acceptable minimun.

5.5. Conclusions on characteristic load models

It has been shown here that the characteristic load models prescribed in EC 1-3 are a good compromise between simplicity and accuracy . The most relevant aspects concerning the application of theses models can be summarised as follow :
- no dynamic effect is to calculate, because this effect is included in the loads.
- a minimum uniform distributed load is applied on all the carriage way, apart lane 1 :
  $q_2 = q_3 = q_r = 2{,}5\ \mathrm{kN/m^2}$,
- a high uniform distributed load, corresponding to a jam of lorries is applied on one lane, 3 m wide : $q_1 = 9\ \mathrm{kN/m^2}$,
- two axle loads are applied on a maximum of 3 lanes with each axle load equal respectively to
  $Q_1 = 300$ kN, $Q_2 = 200$ kN and $Q_3 = 100$ kN.
- in order to avoid local weak points, one axle of **400 kN** (LM2) is to consider alone, every where on the carriage way.

Figures 7 to 10 illustrate the accuracy of the model regarding all the traffic scenarios considered in the calibration, when the heaviest motorway traffic recorded in Europe and an average roughness of the pavement are considered. The code allows also reduction factors β if the expected traffic is not so heavy. Bisedes, when heavier traffics may occur, axle loads should to be considered on more than 3 lanes and high distributed loads on several lanes should be considered.

6. Infrequent and frequent loads

6.1. Definitions

The bridge design needs for the verification of serviceability limit states, the definition of loads that have return periods below 1000 years. For code purposes, the infrequent loads has been defined as having a return period of one year and considering a reduced dynamic effects, corresponding to a good roughness of the pavement.

The frequent loads have been defined as having a return period of one week and considering a good roughness of the pavement and free flowing traffics. The extreme traffic scenarios considered for determining the characteristic loads have not been envisaged here.
The infrequent and the frequent loads may be deduced from the characteristic loads. It has been demonstrated that the load distributions, as well as the load effect distributions, present two modes, and correspond to a Gaussian law for values above the 2nd mode x_o [8] [9]. The value corresponding to a return period R is given by : $x_R = x_o + \sigma . z_R$ (see section 2.1). The ratio x_o/x_k corresponding to free flowing traffics is comprised between 0,3 and 0,5, while 0,7 may be reached for congested traffics. Here only 1 % of the total traffic volume is assumed to run in jam.

6.2. Infrequent loads

For a return period of 1 year, x_R/x_k = 0,9 for free traffics and 0,92 for congested traffics. When a good roughness of the pavement is considered instead of an average roughness, the loads may be reduced by 10 %, so that finally, the infrequent loads in Eurocode are obtained by applying a factor $\mathbf{\Psi'_1 = 0,8}$ on the characteristic loads.
This means that the return period chosen for the definition of characteristic loads is not very important (section 5.1).

6.3. Frequent loads

For a return period of one week and free traffic, x_R/x_k = 0,82. Here too, a good roughness of the pavement allows a reduction of the loads equal to 10 %. But, as the frequent loads result from free traffics only, the uniform distributed loads are always below 50 % of the congested traffic loads [9].
Finally, the frequent loads prescribed by the Eurocode are obtained by applying two different ψ_1 factors on the characteristics values of LM 1 et LM 2 :
ψ_1 = **0,7 for** axle loads and
ψ_1 = **0,40** for distributed loads.

7. Fatigue loads

7.1. Introduction

The calibration of fatigue load models considers free flowing traffics on the slow lane, in fact :
- the fatigue damage concerns mainly short span elements, where dead load is low, and therefore the stress ranges are high,
- on short span elements, below 30 to 50 m, free traffic produces higher load effects than congested traffics (see section 5.2.),
- the highest fatigue damage occurs when the distances between lorries correspond to free traffic [10],
- the highest volume of the traffics runs flowing and not in jam,
- minimum 90 % of lorries are running on the slow lane.

The available data show that the number of lorries on the slow lane of highways is very high, and reaches 1000 to 8000 per day. That will say 25 to 200 million during a life time of 100 years. It results in local elements much more cycles than corresponding to the cut off limit

prescribed in EC 3 (100 million). In order to avoid fatigue damage in bridges submitted to high density traffic, all stress ranges have to be below the fatigue limit under constant amplitude. Therefore a fatigue frequent load has been defined, as a load producing a stress range $\Delta\sigma_f$, in such a way that 99 % of the total fatigue damage results from the stress ranges below $\Delta\sigma_f$. For the fatigue life assessment, an equivalent load has been defined as the centre of gravity of the damage distribution obtained applying the Miner rule [8] [10].
Starting from these considerations, 5 fatigue load models are defined in EC 1-3:
- FLM 1 defines frequent loads derived directly from the characteristic loads by applying two factors : 0,7 on the axle loads of model 1 or 2, and 0,3 on the uniform distributed load.
- FLM 2 defines frequent loads by a set of 5 lorries characterised by the weight, the position and the contact area of each axle, because FLM 1 is not accurate enough for short spans (Figure 11)
- FLM 3 defines a symmetrical vehicle usable for the fatigue life assessment, where the equivalent load of each axle is equal to 120 kN, dynamic effect included,
- FLM 4 defines equivalent loads for the same set of lorries given for FLM 2, allowing a more accurate fatigue assessment than FLM 3, for local effects,
- FLM 5 is not really a load model : a whole load spectrum should be used for a fatigue assessment by applying a cycle counting method and the Miner rule.

7.2. Accuracy of the load models

The fatigue assessment has been performed by considering the free flowing Auxerre traffic recorded on the slow lane, and SN curves with 3values of the slope, corresponding to m = 3, 5 and 9. In Figure 11 the ratio between $\Delta M_{fEC\,1}$, which are the effects produced by FLM 1, and ΔM_{fA}, which are the target effects produced by the Auxerre traffic, is given, depending on the span, for m = 3. The Figure shows that FLM 1 gives too high values for short spans (L < 20 m.), and too low values for one influence line (M_2). The first problem is solved by FLM 2 (see Figure 12). The second problem should be solved by increasing the uniform distributed load, for example by accepting here the frequent load defined in section 6.3. FLM 1 and FLM 2 have to be on the safe side in all cases, because, if these models show that the fatigue life is limited, the final conclusion of the fatigue assessment results from the use of FLM 3 or FLM 4.

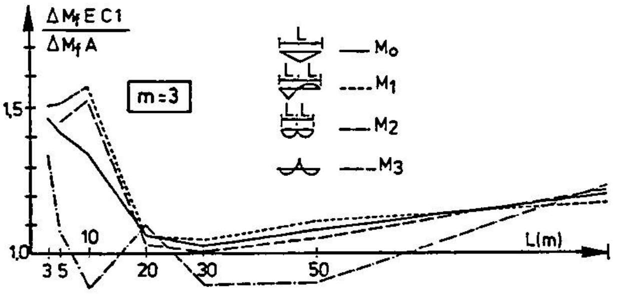


Fig. 11 FLM 1

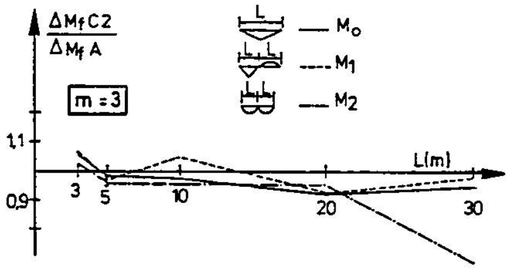


Fig. 12 FLM 2

Figure 13 gives the ratio between ΔM_{eEC},which are load effects produced by FLM 3 and ΔM_{eL}, which are the equivalent load effects produced by the Auxerre traffic, where the equivalent number of cycles is given by : $\mathbf{n_e = k_1.k_2.n_L}$, where

$k_1 = 2/3$ for Auxerre traffic,
$k_2 = 1$, if< 10 m. ,
$k_2 = 0{,}6 + 1/0{,}25$ L, if $1{,}18$ m. $\leq$ L $\leq$ 10 m,
$k_2 = 4$, if $L \leq 1{,}18$ m. ;
L is the span length. ;
n_L is the number of lorries.
The ratio is generally between 0,95 and 1,15, if the load effect on support M_2 is disregarded . In order to solve the problem of M_2 when FLM 3 is used, it is necessary to consider a **second vehicle** 40 m. after the first. The second vehicle has the geometry of FLM 3, while the axle loads are multiplied by a factor 0,3 (see Fig. 14).
The need of a second vehicle, running 40 m. after the first, results from the analysis of the traffic and from the shape of the influence lines :
- the probability of the presence of 2 vehicles on a lane length longer then 40 m. is significant,
- the second vehicle increases the equivalent load effect in span (M_0, M_1, M_3) for spans longer than 80 m., and on support (M_2) for spans longer than 25 m. Practically, the second vehicle is only needed for the fatigue assessment of details where the influence line presents two contiguous areas of the same sign.

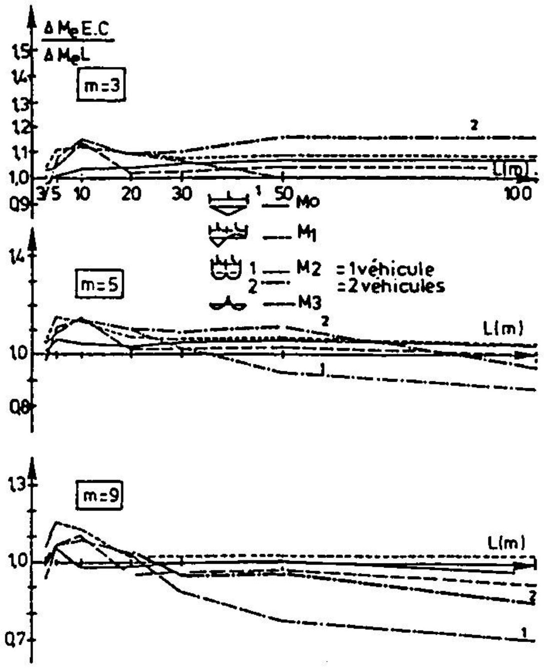


Fig. 13 FLM 2

In conclusion, the models prescribed in EC 1-3 result very accurate and independent on the slope of the SN curves defined by the factor m.

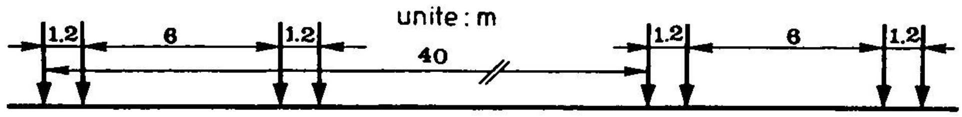


Fig. 14 F LM3 - modified.

7.3. Fatigue assessment using damage equivalent factor λ

Fatigue assessment can be also carried out, using the so called damage equivalent factor λ.
The basic idea of the λ factor method is to relate the damage induced by the stress spectrum to an equivalent stress range $\Delta\sigma_{eq}$ referring to 2 x 10^6 cycles, $\Delta\sigma_{eq} = \lambda.\varphi_{fat}\,.\,\Delta\sigma_p$, where $\Delta\sigma_p$ is the maximum stress range induced by the fatigue load model $\Delta\sigma_p = (\sigma_{p.max} - \sigma_{p.min})$ and φ_{fat} is the damage equivalent impact factor.
Of course, the λ factor depends on the material by the slope m of the S-N curve.
When the fatigue assessment is based on FLM 3, the damage equivalent factor can be expressed as $\lambda = \lambda_1.\lambda_2.\lambda_3.\lambda_4$, where λ_1 depends on the shape and on the length of the influence line, i.e. on

traffic flow and the traffic composition , λ_3 is a factor depending on the design life of the bridge and λ_4 takes into account the multilane effect.
The numerical values of λ_i depend, as well as on the slope m, on the reference traffic used for the calibration and on the reference design life of the bridge LT_R. Said N_o the flow and Q_o the equivalent weight of the reference traffic, it is

$$\lambda_2 = k.\frac{Q_{m1}}{Q_o}.\left(\frac{N_1}{N_o}\right)^{1/m}$$, where k is a constant, N_1 is the actual flow and

$$Q_{m1} = \left(\sum_i n_i Q_i^m / \sum_i n_i\right)^{1/m}$$, the equivalent lorry weight for the considered lane,

$$\lambda_3 = (LT/LT_R)^{1/m}$$, being LT the actual design life.

To evaluate λ_4 it is necessary to take into account, as well as the effect of the lorries travelling alone on different lanes, the simultaneous transit of lorries on several lanes [11] , so that

$$\lambda_4 = \left\{\frac{N_1^*}{N_1} + \sum_i\left[\frac{N_i^*}{N_1}.\left(\frac{\eta_i}{\eta_1}\right)^m\right] + \sum\left[\frac{N_{comb}}{N_1}.\left(\frac{\eta_{comb}}{\eta_1}\right)^m\right]\right\}^{\frac{1}{m}}$$, in which

N_1 is the lorry flow on the main lane, η_i is the effect of the i-th lane, N_i^* is the flow of the individual lorries on the main lane, N_{comb} the flows and η_{comb} the effects of interacting lorries, and the second sum is extended to all the relevant combinations of several lorries.

7.4. Conclusions on fatigue assessment

The fatigue load models defined in E 1-3 allow a simple approach of the fatigue assessment using the SN curves of the detail to verify [12].
If a fatigue limit under constant amplitude is defined, as in the design code for steel structures EC 3, the use of the frequent load models FLM 1 or FLM 2 may allow a first quick conclusion concerning the existence, or not, of a fatigue damage.
The fatigue life may be calculated by using **FLM 3**, if two requirements are satisfied :
1. the SN curves are unlimited : the cut off limit defined in EC 3 have to be deleted,
2. two vehicles have to be considered with a spacing equal to 40 m.
FLM 2 and FLM 4 are more accurate only for the fatigue life assessment of local effects, occurring in concrete or orthotropic slabs.
It could be suggested to increase the values of FLM 1 up to the frequent values given in section 6.3.

8. Conclusions

Starting from a wide set of data obtained by in site measurements of road traffic loads, it has been possible to define scientifically the representative values needed for design of bridges. The main load models given in the Eurocode 1-3 have been calibrated on a Continental European highway traffic. In order to take into account lighter traffics reduction factors are foreseen, while loads can been incresead when heavier traffics are expected.
The fatigue load models allow an engineer approach by checking first if fatigue damage is expected or not, and than by calculating the fatigue life.

The aim of the drafting panel of EC 1-3 was to propose models that are a good compromise between accuracy and simplicity, in spite of the complexity of the problem.

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