Remaining fatigue life of bridges

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Remaining Fatigue Life of Bridges

Durée de vie résiduelle des ponts

Restlebensdauer von Brücken

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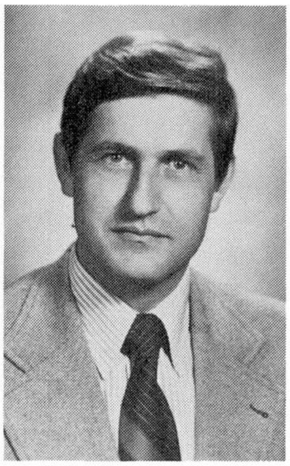

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SUMMARY
The evaluation of the remaining fatigue life of an existing structure involves the following important steps. Firstly, two load models, representing past load history and future traffic have to be established. Then the static and dynamic structural performance must be assessed by either computer analysis or in-situ stress measurement or both. Having also established an appropriate fatigue strength curve, the theoretical remaining fatigue life may be evaluated using probabilistic methods. These assessments are becoming increasingly more important as many existing structures are exceeding their design lives.

RESUME
L'estimation de la durée de vie résiduelle d'une structure existante soumise à la fatigue comprend les principales étapes suivantes. En premier lieu il faut établir deux modèles de charges, l'un représentant les charges antérieures supportées par l'ouvrage et l'autre la prévision du trafic à venir. Il faut ensuite déterminer le comportement statique et dynamique soit par une analyse à l'aide de l'ordinateur, soit par les deux moyens. Après avoir également choisi la courbe de fatigue appropriée, la durée de vie résiduelle théorique peut être déterminée à l'aide de méthodes probabilistes. Ces évaluations deviennent d'autant plus nécessaires que de nombreux ouvrages existants ont dépassé leur durée de vie prévue lors du dimensionnement.

ZUSAMMENFASSUNG
Die Abschätzung der Restlebensdauer von bestehenden Konstruktionen umfasst die folgenden wichtigen Schritte: Zuerst werden zwei Lastmodelle aufgestellt und zwar einerseits für die Lastgeschichte und anderseits für den zukünftigen Verkehr. Dann muss das statische und dynamische Tragverhalten durch Computersimulation und/oder durch Spannungsmessungen am Objekt erfasst werden. Liegen die relevanten Ermüdungsfestigkeitswerte ebenfalls vor, kann dann die theoretisch vorhandene Restlebensdauer mit Hilfe von Wahrscheinlichkeitsüberlegungen abgeschätzt werden. Dieses Vorgehen gewinnt zunehmend an Bedeutung, da viele bestehende Konstruktionen ihre Bemessungslebensdauer schon überschritten haben.

1. INTRODUCTION

There are different reasons why an evaluation of the remaining fatigue life might become necessary. The most obvious need occurs when cracks are found in a structure. Another reason for evaluation arises when significant changes have happened during the life of the structure. A third and economically important aspect, particularly due to the large number of cases involved, concerns structures approaching their theoretical design life.

This paper tries to identify the basic parameters needed for the evaluation of the remaining fatigue life. Each parameter is discussed and its data base and importance in the evaluation is considered. Based on this and using commonly accepted rules for cumulative damage, simplified methods of evaluation are shown. It must be added that generalized rules are neither available nor have been agreed upon, as yet.

It should also be recognized that one of the most important aspects of the evaluation procedure is the insight into the problem, and the ensuing possibility for correctly rating the structure. Deterministic approaches are generally used, sometimes introducing statistical values for the fatigue strength. More research is under way to establish clear lines for assessing the probability of survival using modern safety concepts. Such procedures are hindered by lack of knowledge of the effects of loading and the need to calibrate with experience.

A closely related problem is the rating of a complete set of structures, for example all railway bridges on a given stretch of line, or all highway bridges in a county or state. This aspect will become more and more important since the number of "old" bridges increases every year. Therefore, decisions have to be made whether to keep these bridges in service beyond their theoretical design life, to replace them, or to strengthen them. Priorities to carry out this work also need to be established.

2. MOTIVATION AND GOALS

The main purpose of the evaluation of the remaining fatigue life resides in the rating of the structure. This rating has to include decisions on various actions such as inspection, retrofit, repair, strengthening and replacement of elements or even the whole structure.

There are three distinct circumstances where such a rating is needed :

1.- cracks are found in a structure,
2.- the structure approaches its design life,
3.- it is recognized that important changes have occurred.

In the first case, immediate action has to be taken in order to decide whether or not a structure has to be closed to traffic. The investigation, very often based on modern methods of fracture mechanics, will reveal what type of repairs or retrofit procedures are needed to keep the structure in service.

The second case involves an increasingly large number of structures. New codes commonly define design lives of the order of 30 years for crane gantry girders, 50 years for highway bridges, and 100 years or more for railway bridges. The public and even many engineers relate these ("arbitrarily" chosen design) values to existing structures, although they may never have been designed for fatigue. Approaching this design life is often equated to an "unsafe" condition. As a consequence, transport authorities have to define priorities in the replacement of these "overdue" structures, or produce evidence that they may be kept in service. The tendency, for economic reasons, is to hang on to existing structures unless other conditions such as maintenance problems or operational requirements become predominant.

The third case encompasses a large variety of structures which have experienced major changes. These changes are not always obvious and they may be of quite different natures, such as :

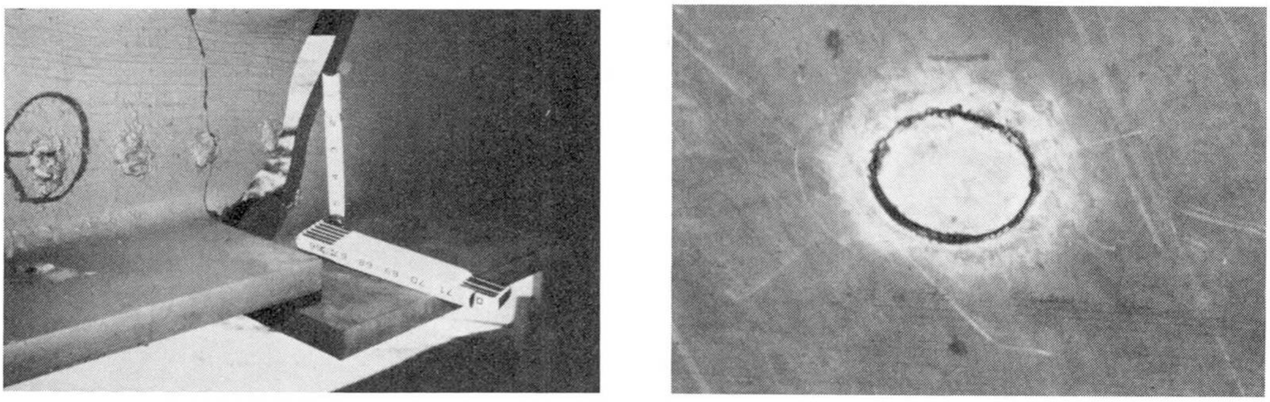
- physical modifications to the structure,
- improvement of knowledge,
- increase in traffic.

Physical modifications may include :

- changes due to fabrication and erection procedures which were not accounted for in the design. Welded lifting attachments left in place, bolt holes or flame cut notches filled with weld material, cut-out elements rewelded in place (FIGURE 1), etc. ;
- changes due to strengthening or widening of a structure in order, for example, to accomodate increased traffic volume or loads ;
- attachments added to hold utility lines (gas, water, sewer, etc.) ;
- replacement or repair of corroded elements or parts thereof by, for example, fillet-welding doubler-plates.

Improvement of knowledge recognizes the fact that new insight has been gained into the fatigue behavior of structures. For example, better information on the fatigue strength of typical welded details is now available todate than compared with 20 to 30 years ago. In countries where modern fatigue clauses have not yet or only recently been introduced, it is quite probable that severe details have been built into the structures. These details were not recognized as serious at the time of the design, in the same way that physical modifications are often not recognized to be detrimental. In addition, the widespread introduction of high strength steels and new welding procedures (electroslag welding) was sometimes undertaken without much knowledge of their behavior and performance. Design rules were mainly based on static strength.

The most widely recognized change, but not always the most important, is the



a) Fatigue crack emanating from bolt holes filled with weld material.

b) Flame-cut plate element before welding back into its original position.

FIGURE 1 : Examples of the possible effects of fabrication and erection procedures.

increase in traffic over the past twenty years. One of the most disturbing aspects of this observation is the great difficulty to model future traffic, both in load intensity and traffic volume. Connected to that is the question of how the change of legal load limits will affect the remaining fatigue life of structures.

Before discussing the different parameters and assumptions needed for an evaluation of the remaining fatigue life, it is necessary to point out that no unique or generally applicable method exists or has been agreed upon. The methods used should necessarily reflect the specific goals for the given structure. Considering also the time and money available, it would appear sensible to :

- proceed in steps, going from a rough approximation to more detailed and refined approaches (different levels) ;
- start from the safe side, that is overestimating stresses and underestimating strength ;
- rapidly conclude, whether a problem exists or not, and only if there is an indication of a possible problem proceed to a higher level of approximation.

In proceeding this way, one may also obtain an idea of the influence of improved assumptions on the resulting estimate of the remaining fatigue life. This appreciation of sensitivity may be useful in the judgement of the result and, hence, in the rating of the structure.

The evaluation and the ensuing rating, irrespective of motivation, should identify the most critical elements within a given structure, provide guidance for inspection intervals during the remaining fatigue life and allow priorities to be established for replacement or inspection in a given set of structures. Another important goal is to forsee an answer to the economic impact due to the possible increase of legal load limits.

3. BASIC PARAMETERS

The following section tries to identify possible steps, or levels of precision, in the definition of significant parameters. It is generally done by starting with simplified assumptions before going into detailed considerations, which require a large amount of calculations, or the evaluation of statistical data, or even tests.

3.1. Load History

Step 1 : Traffic model based on present traffic.

A fatigue load model may consist of a set of typical load cases described by the disposition of the loads and their intensities including the relative occurence of each load case. Such load models have been proposed by a few modern codes or specifications [1] [2] [3]. FIGURES 2 and 3 show the fatigue load models [4] [5] implicitly used in the Swiss steel specifications. Both models give the load intensities and geometries of trains or trucks and the relative occurrence of the different types. A comparison with the actual traffic and the use of these fatigue load models is further described in paragraph 3.8.

Step 2 : Evaluation of past traffic conditions.

Changes in past traffic may have occurred at different periods of time during the life of the structure, either gradually or rather quickly. One might consider for example :

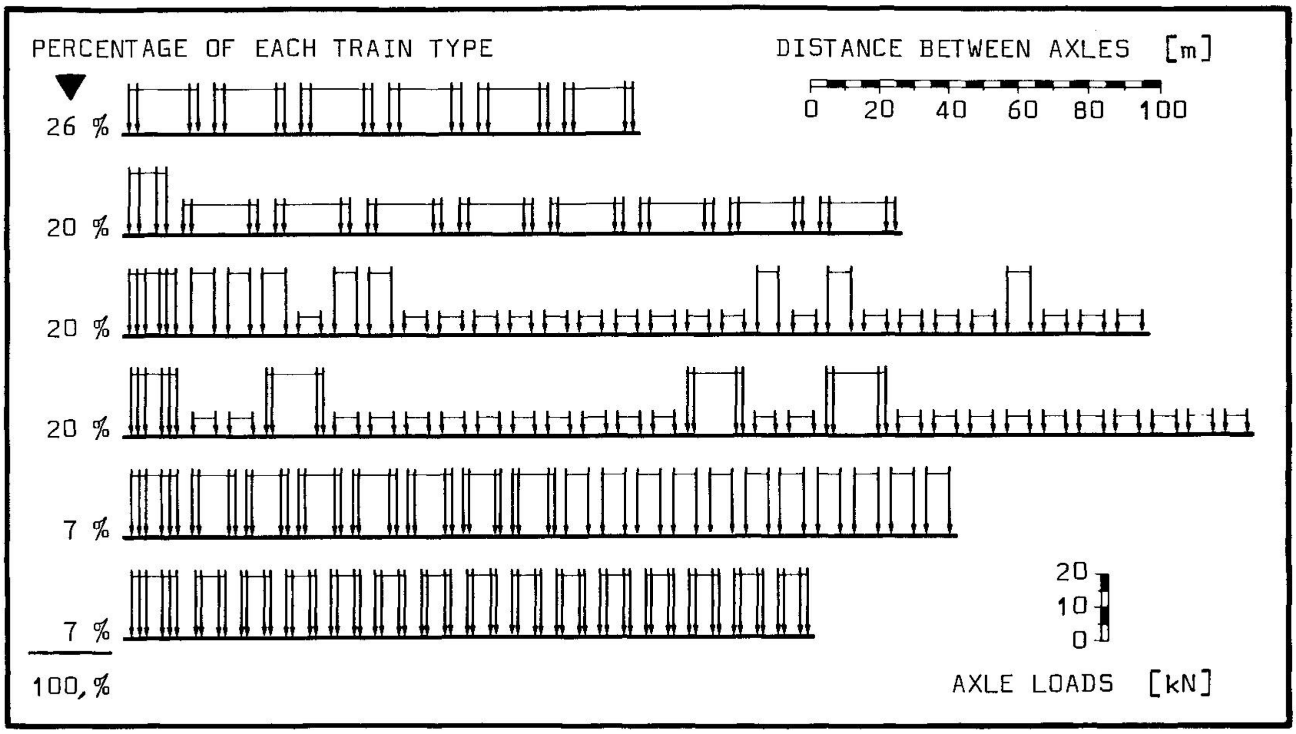


FIGURE 2 : Fatigue load model (Swiss Federal Railways) representing actual traffic on railway bridges.

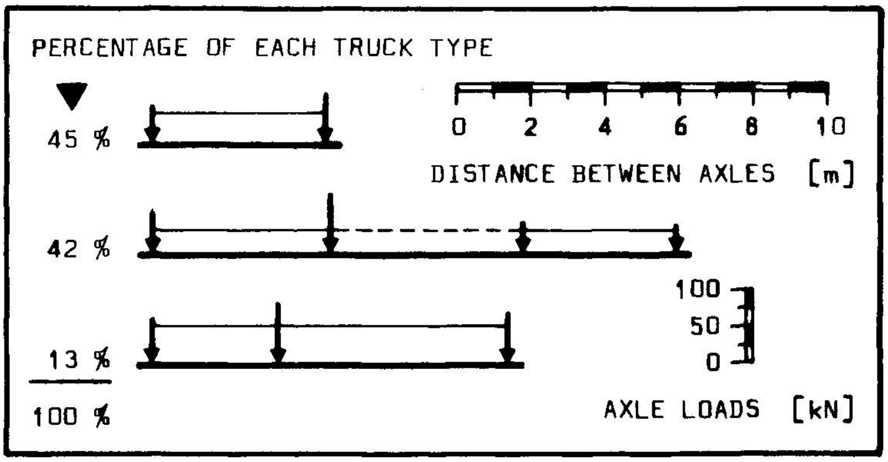


FIGURE 3 : Fatigue load model representing actual traffic on Swiss highway bridges.

- technical development of the vehicles, such as at the time of electrification of railway lines, or use of diesel locomotives ;
- change in traffic pattern when new lines or highways are opened ;
- effect of war-time loads ;
- change of design codes or maximum legal limits on loads, which incidentally might have resulted in strengthening the structures.

3.2. Future Loadings

Step 1 : Present traffic situation.

Using the present traffic situation to describe future loading, therefore neglecting possible increases, does obviously not represent a conservative assumption, but it has the merit of being simple. It might even be accurate enough when the calculated remaining fatigue life is short. If this should not be the case, one might then still proceed to an educated guess of the future traffic situation.

Step 2 : Estimated future loads.

Traffic development, particularly the increase of axle loads or total truck or car weight, is likely to be influenced by political decisions and economic factors. One example is the effect of Common Market agreements in Europe which tend to adjust legal load limits in the various countries. It is apparent from FIGURE 4 that the legal load limit influences directly the position of the peak in the probability density functions of heavy truck traffic [6]. Another pressure calling for heavier truck weights comes from the ecology movements and fuel efficiency in order to reduce the number of trucks needed to transport the same total tonnage.

Step 3 : Extreme load situation limited by maximum capacity or operational limits.

It should be noted that steps 2 and 3 are open guesses, particularly if the traffic evolution over a period of 20 years or more must be estimated.

As a conclusion, it is preferable to operate on a resonable level of knowledge, for example the present traffic condition including scheduled increases. This type of evaluation might appear not to be on the safe side. However, safeguards can be provided by specifying the traffic conditions for which, when reached, a renewed evaluation becomes obligatory.

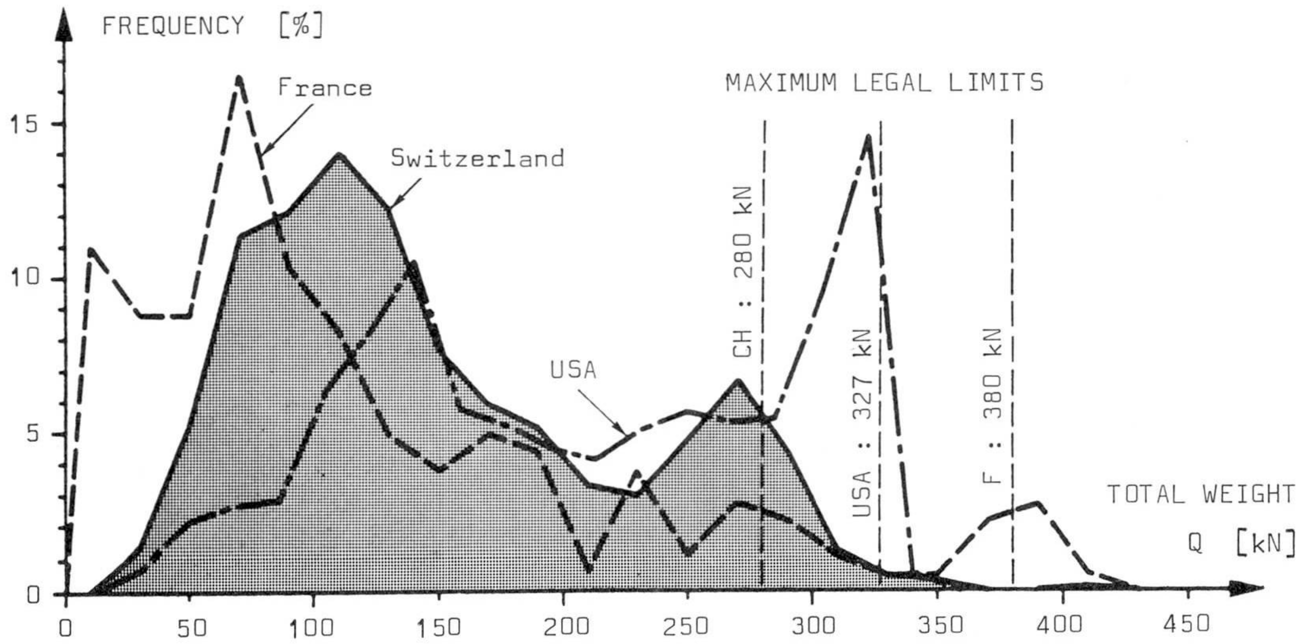


FIGURE 4 : Comparison of truck weight histogrammes from three contries.

3.3. Static Stresses and Strains

Step 1 : Simple stress analysis according to common engineering practice.

Step 2 : Detailed analysis based on computer methods.

Step 3 : In-situ measurements under well defined loads.

Generally, a difference is observed between measured and calculated stresses. This difference may vary from one element to another, and from one point to another depending on the type of structural system. Also, the difference becomes smaller with improved approximation of the structural system.

The measured stresses at midspan are generally smaller than computed although this does not hold for the support region. Typically, a floor beam calculated as a simple beam might behave more like a fixed-ended beam under service conditions due to the structural detailing of the support region, even though at ultimate load the statical system is more like a simple beam.

Hence, support regions are likely to be more highly stressed that calculated. In addition, they might impose stresses and strains to the supporting elements, which might lead to strain induced cracking [7] [8].

Finally, one must check whether changes have occurred in the static behavior of the structure or its elements. This may be due to strengthening of the structure, changes in the superstructure, support settlements, "frozen" bearings due to corrosion or dirt, etc.

3.4. Impact Factor for Dynamic Behavior

Step 1 : Impact factor according to design codes.

Step 2 : Information based on experimental evidence from similar structures.

Step 3 : In-situ measurements of the live load stresses.

Design impact factors, as a rule, are on the high side ; this is particularly true for short span elements. Some codes, like UIC [9], give additional values for the assessment of bridges introducing for example parameters such as : length of the influence line, natural frequency, track condition. It is obvious that a ballasted railway bridge deck will have less dynamic impact than when the rails are directly connected to the structural system composed of floor beams and stringers.

Similarly, the road surface condition affects dramatically the impact values for highway bridges, particularly when pot holes are present. Jumps at expansion joints or those due to the settlement of the approach slab may also impose additional dynamic effects. As a consequence, and due to the relatively great influence of impact factors on the result of the evaluation, measurements may often be very useful.

3.5. Constant Amplitude Fatigue Strength

Step 1 : Assumptions based on design curves.

The most important aspect is the correct identification of the severe details and their relationship to a given classification system. FIGURE 5 shows S-N curves proposed by ECCS [10] with a double-logarithmic scale. Having parallel lines

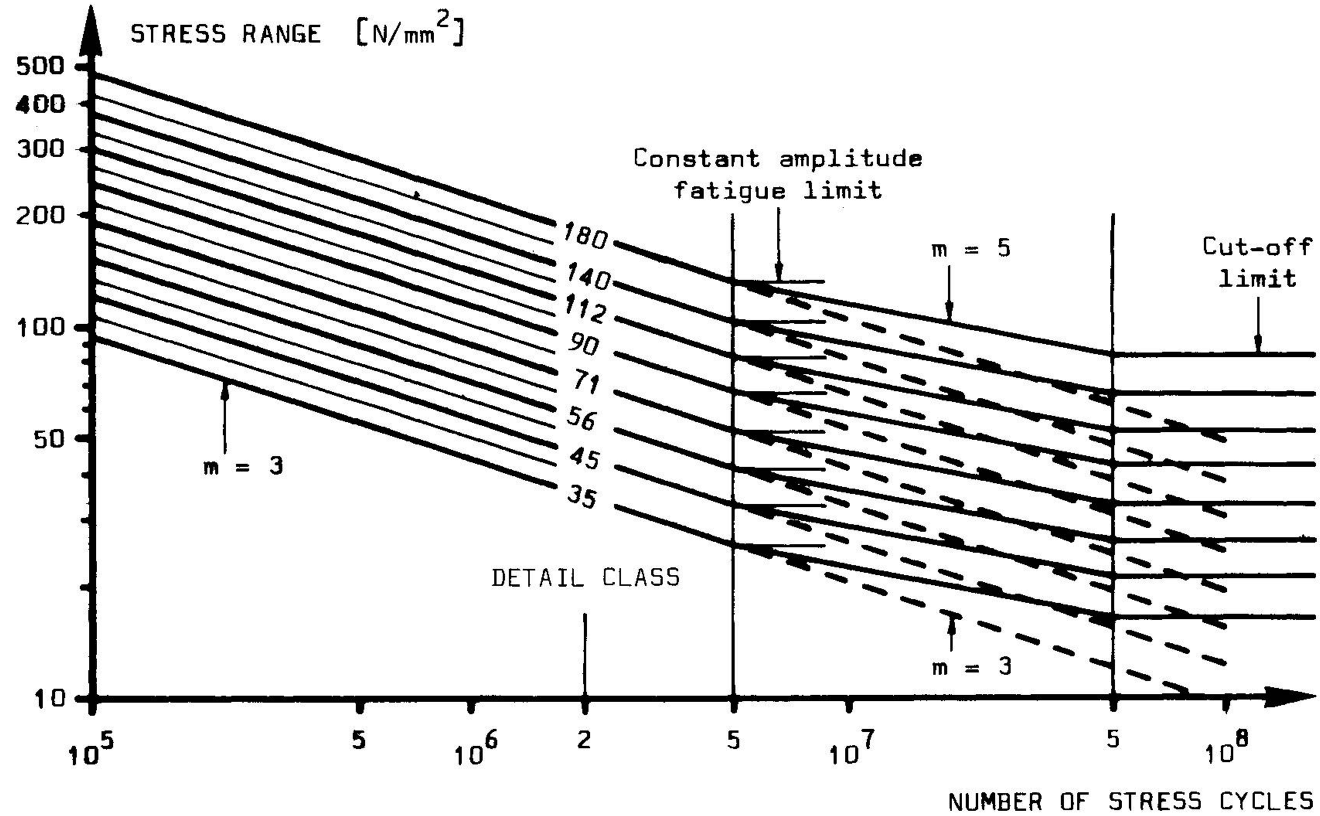


FIGURE 5 : **ECCS proposal for "European Fatigue Strength Curves" representing mean minus two standard deviations.**

greatly simplifies the designers work and the definition of equidistant curves sets the level of accuracy needed. This is particularly important in avoiding over-precision of parameters of lesser importance.

It should be noted that stress range is the governing factor for the fatigue life of a given structural detail. Minimum stress, stress ratio, and even grade of steel do not significantly affect the fatigue strength [11].

It is possible that particular details of a given structure will not be identified in a classification system. It should be possible, based for example on fracture mechanics considerations, to conservatively introduce such details in the system. Important parameters for the evaluation of details are stress concentration due to the general stress field and defect size.

Special attention has to be paid to built-in defects. Such defects are commonly created by incomplete penetration welds at the crossing of different (secondary) elements. This type of defect, where the lack of penetration is perpendicular to the stress field, is not contained in the usual classifications systems.

Step 2 : Use of published data.

When using test data, for example for old riveted structures, one should recognize that test data obtained with small test specimens will overestimate the fatigue resistance. The effect of grade of steel is often presented on the basis of machined base material specimens which obviously do not reflect the stresses and notch conditions of real connections.

Another parameter, historically important since most early design codes used it, is the mean stress or stress ratio. In practice, this effect is generally no longer considered since :

- the mean stress in the structural element is not known due to the influence of thermal stresses, stresses due to misfits, or effect of support movements ;
- the fatigue strength curves for the various mean stresses are not available ;
- large welded elements simply do not show the effect of mean stress ;
- cumulative damage rules considering mean stress are not well established.

Step 3 : Tests on structural elements.

In case of the evaluation of a large set of structures, for example riveted bridges built before the turn of the century using wrought iron, this approach might be justified. It is sometimes possible to remove typical details from an existing structure or use material from a similar structure being dismantled. When interpreting the test data, it is important that fractographic examinations are made in order to check whether small fatigue cracks had already existed in the test specimens at the onset of the tests.

3.6. Counting Method

Step 1 : Major stress cycles only.

The fact that stress range is the predominant parameter for fatigue strength implies that stress ranges have to be identified in a given stress-time diagram, be it calculated or measured. In the first step, the major stress ranges are counted using for example peak counting (might be overconservative) or peak-to-peak counting. All stress ranges smaller than about 30 % of the major stress ranges can be neglected, provided their number (frequency of occurrence) is of the same order of magnitude. This can be verified using the equivalent stress range concept given by Equation 2 in paragraph 3.7.

Step 2 : Rainflow counting (Reservoir method).

Rainflow and range-pair count theoretically give the same results provided that the level of the neglected stress cycles for the range-pair count is kept very small. This indicates one advantage of rainflow, where the decision on the suppression of small cycles is not needed before the counting. On the other hand, the computer programming of rainflow is not very convenient. Also, one has to be aware that many rainflow programs are not prepared to handle stress excursions with changing sign, as for example for the influence line of a continuous beam. For manual counting the representation using the reservoir model is more attractive.

Note : In order to have a common basis for comparison and discussion, ISO, Eurocode, ECCS and IIW propose to use rainflow counting [12] [13] [14] [15].

3.7. Cumulative Damage Calculations

It is proposed and assumed that cumulative damage will be calculated according to Palmgren-Miner's rule equating the total sum of damage to unity. If this is not done it should be clearly stated. It is also important to indicate which fatigue strength curve (mean or %-fractile) has been used for the cumulative damage calculation.

If all stress cycles fall below the fatigue limit, it is assumed that no fatigue damage occurs or has occurred [14]. When the stress spectrum is such that a part of it lies above the fatigue limit, three steps of approximation are possible :

Step 1 : The fatigue limit is disregarded.

All stress ranges are considered to be fatigue damaging. Based on the equation of the fatigue strength curves,

$$N = C\,\Delta\sigma^{-m}\ , \tag{1}$$

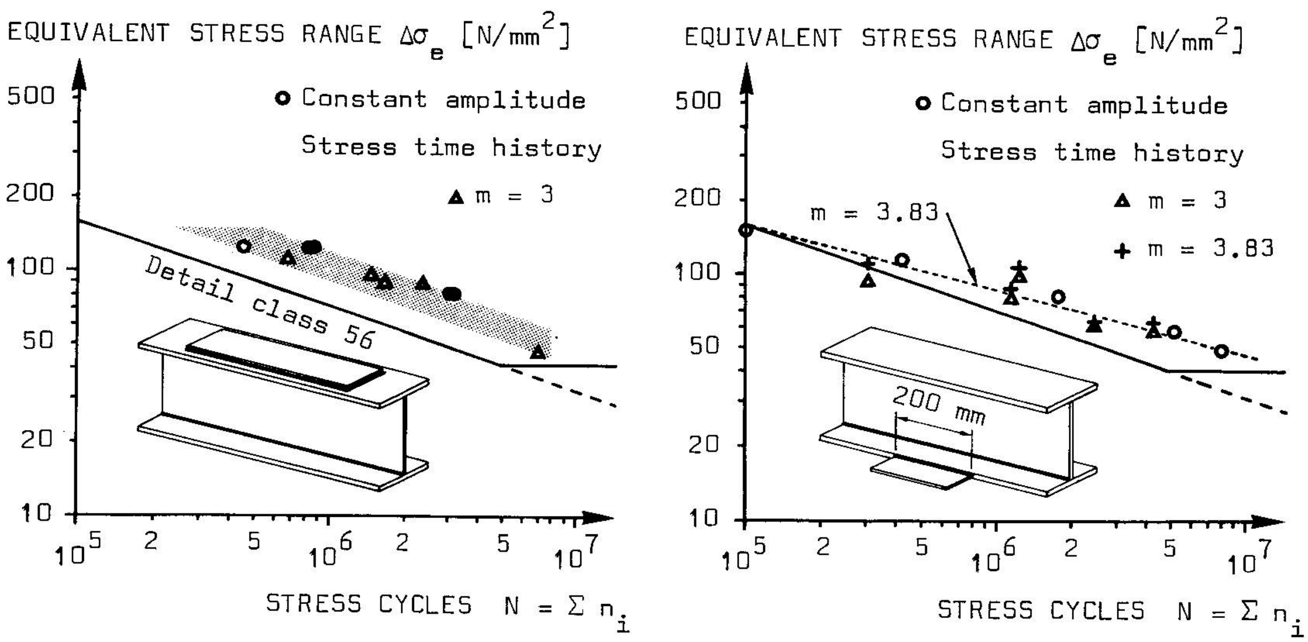
and using Palmgren-Miner's rule, it is possible to express the stress spectrum by an equivalent stress range $\Delta\sigma_e$ [16], which would yield the same number of constant amplitude cycles $N = \Sigma\, n_i$ as contained in the stress spectrum :

$$\Delta\sigma_e = \left[\frac{\Sigma\, n_i\, \Delta\sigma_i^m}{\Sigma\, n_i}\right]^{1/m} . \tag{2}$$

This has been experimentally verified on a large series of test beams submitted to programmed loading [17]. With these programmed loadings, the validity of the equivalent stress range, or in other words, the cumulative damage rule is checked.

True stress-time histories do not correspond to block loadings or to random loadings. In addition, a counting method has first to be used in order to identify each stress range cycle. Pilot studies on test beams subjected to recorded stress time histories from either highway traffic or railway traffic have shown that the equivalent stress range may be used in conjunction with rainflow counting (FIGURE 6). A parametric study is under way to evaluate the effect of other counting methods and mean stress.

FIGURE 6 shows that, for purposes of design or evaluation of the remaining fatigue life, the equivalent stress range concept is quite adequate. It is noted (FIGURE 6 a) that the scatter of the stress history data is about the same as



a) Coverplated beams.

b) Beams with gusset plates.

FIGURE 6 : Data from fatigue tests with stress-time histories analyzed by rainflow counting and equivalent stress range (detail class 56 denotes the fatigue strength at $2 \cdot 10^6$ cycles and is identical to AASTHO category E).

that for the constant amplitude data. At any rate, it should not be expected that the variable amplitude data would show smaller scatter than the basic data.

FIGURE 6 b shows that the exponent m (slope of the S-N curve) has little effect on the fit of the data. Even though the observed slope of m = 3.83 gives a better fit for this particular detail and small sample size, the use of the common slope of m = 3 is still satisfactory. Generally, larger test data samples tend toward an exponent of 3 for the lower bound.

Step 2 : Fatigue strength curves with a knee point.

It can be concluded from step 1 that the equivalent stress range concept may be used for a rapid evaluation. However, when a large portion of the stress spectrum falls below the constant amplitude limit, this procedure may give over-conservative estimates. Different ways to account for stress ranges smaller than the fatigue limit have been proposed. A simple approach consists of introducing a bi-linear S-N line with a knee (for example at 5 million cycles) below which a smaller slope (for example (2 m - 1) or (m + 2), as shown in FIGURE 5) is introduced [11] [18]. Tests are under way in various laboratories to further investigate this proposal, which is at present quite commonly accepted.

Step 3 : Fracture mechanics analysis.

Simplified or sophisticated fracture mechanics procedures are generally not recommended for the evaluation of the remaining fatigue life of structures, unless a crack has been observed. In such a case, the location and possibly the dimensions and shape of the crack are known as well as the stress field surrounding it. A detailed analysis then becomes possible or even necessary in order to define retrofit or repair needs.

3.8. Composition of Traffic

Over the past twenty years, extensive research has been carried out on the fatigue strength of details. The description of loads has long been a neglected part of the problem, and it seemed impossible to compare loads in or between different countries. However, it has been shown recently that a comparison is possible provided it is not made on loads (intensity, geometry, etc.) but on their cumulative damage [6] [19] [20].

In order to make a comparison, one needs a well defined reference load which might be the same as the standard live load used for static design. An example for railways is shown in FIGURE 7. The extreme maximum and minimum stresses due to this load are calculated and used to obtain a reference (design) stress range $\Delta\sigma_d$. Before showing examples, it is necessary to recall some important characteristics of the S-N diagram (FIGURE 8) using the equivalent stress range concept.

Assuming that the equivalent stress range $\Delta\sigma_e$ and the corresponding number of stress range cycles $N = \sum n_i$ is known, a damage line can be introduced in

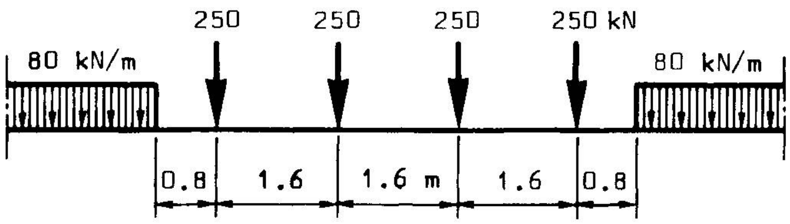


FIGURE 7 :

UIC standard design live load for rail bridges (UIC : Union Internationale des Chemins de fer).

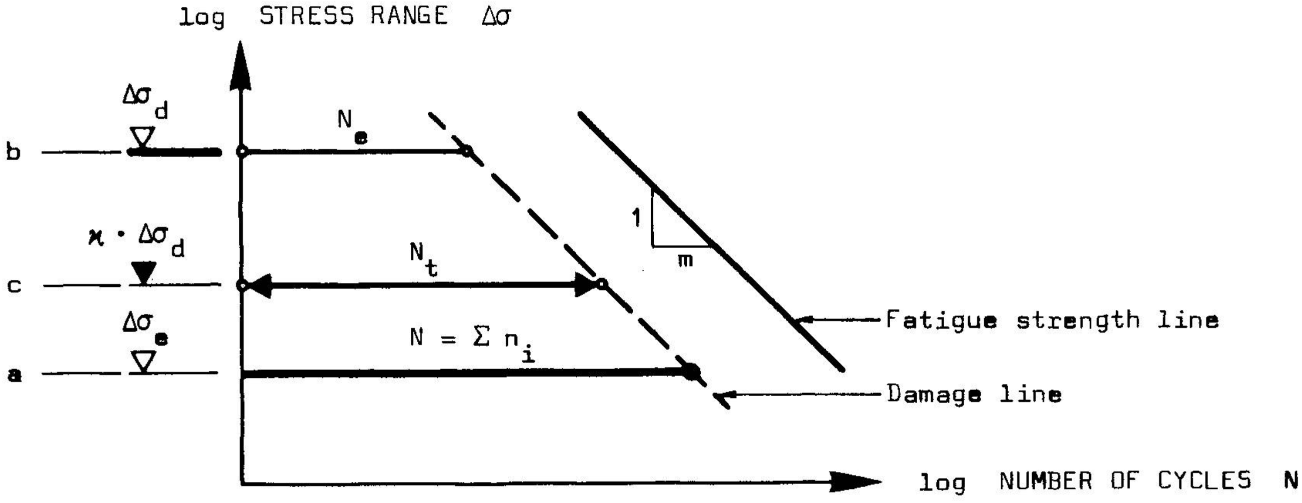


FIGURE 8 : Definition of three levels of stress range with the corresponding number of cycles yielding the same cumulative damage.

FIGURE 8 parallel to the fatigue strength line. Two other points may now be defined on this same damage line. One is fixed at the level of the reference (design) stress range $\Delta\sigma_d$ and the other by a preselected number of stress cycles. Therefore, three distinct levels are defined by the damage line among which certain correlations may be retained :

- Level a is based on the stress spectrum and expressed by its equivalent stress range $\Delta\sigma_e$ (Eq. 2) and the corresponding number of stress cycles $N = \Sigma\, n_i$.
- Level b is defined by the stress range $\Delta\sigma_d$ due to a reference load and related to level a by an equivalent number of stress cycles N_e, where

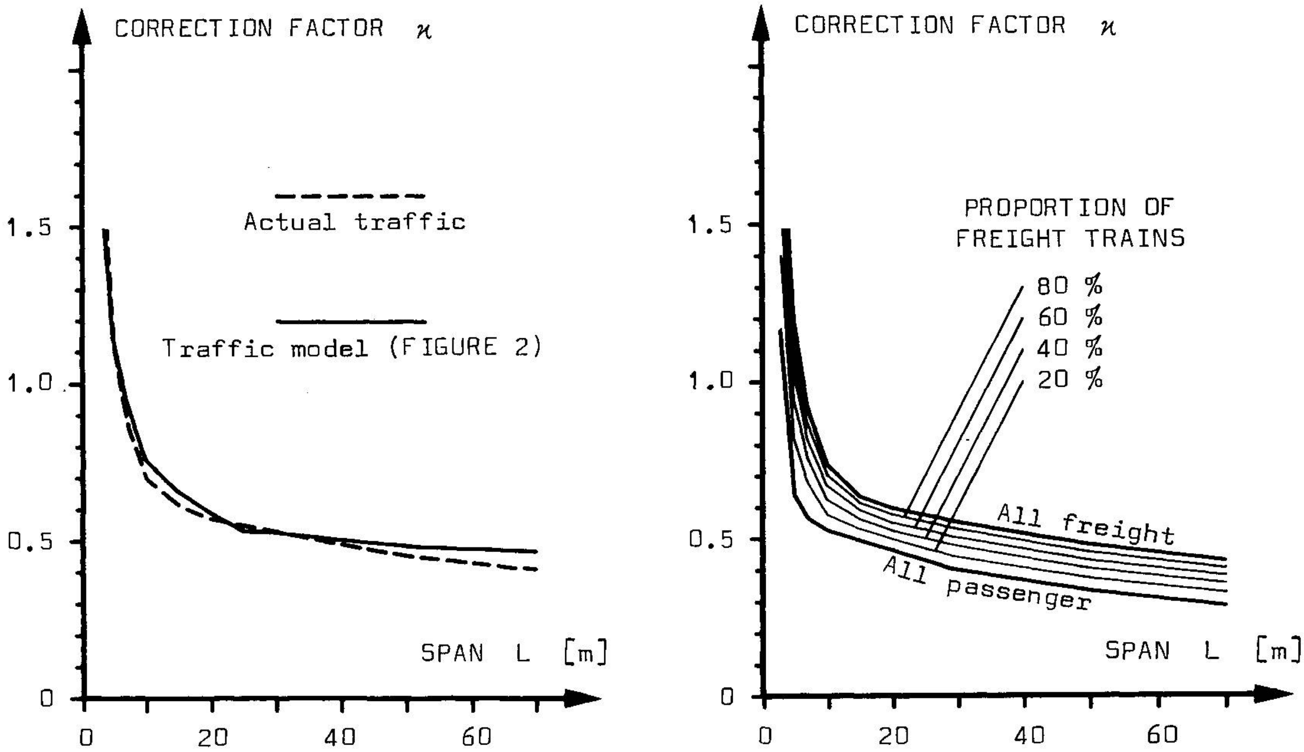
$$N_e = \left(\frac{\Delta\sigma_e}{\Delta\sigma_d}\right)^m N \, . \qquad (3)$$

- Level c is given by an arbitrarily fixed number of cycles N_t. Say for example the number of trains which would lead to the same value for all bridge elements, as opposed to the number of stress cycles which vary with influence length. The corresponding level of $\varkappa \cdot \Delta\sigma_d$ is expressed in terms of a correction factor $\varkappa$, multiplied by the design stress range $\Delta\sigma_d$ for ease of comparison, where

$$\varkappa = \left(\frac{\Delta\sigma_e}{\Delta\sigma_d}\right) \left(\frac{N}{N_t}\right)^{1/m} . \qquad (4)$$

When these relationships are applied to <u>railway bridges</u> [16] [20], it becomes possible to verify whether a load model (FIGURE 2) represents the fatigue effects of real traffic (FIGURE 9 a). Since a common European load model (FIGURE 7) is used, all countries can thus compare their load models or the effect of their actual traffic in one and the same way.

The effect of traffic composition, showing for example different proportion of freight trains, may be studied in the same way. FIGURE 9 b has been established on the basis of about 150 measured trains [21]. Finally, it should be noted that this correction factor has been introduced in the Swiss Steel Specification [1] and UIC Recommendations [3] where it is called α and λ_T, respectively.



a) Comparison of the computed fatigue effects of a traffic model and actual traffic.

b) Effect of the proportion of freight trains in the traffic.

FIGURE 9 : Possible applications of the correction factor $\varkappa$ for railway bridges.

Another example relates to highway bridges [6]. Considering the great difference in the histogrammes of the truck weights for different countries (FIGURE 4), it seems impossible to find a common denominator. Even more so when the differences in type of trucks and their geometry is observed. However, it has been found [5] that the cumulative fatigue damage of a given truck traffic, including the variation in weight and geometry of all trucks, can be expressed by a correction factor.

The procedure is as follows. First, the equivalent stress range $\Delta\sigma_e$ is computed using the stress ranges of all individual trucks. Then, a stress range due to a single concentrated load, represented by the equivalent weight Q_e

$$Q_e = \left[\frac{\Sigma\, n_i\, Q_i^m}{\Sigma\, n_i}\right]^{1/m} \quad . \qquad (5)$$

is also computed. The correction factor is the ratio between the equivalent stress range $\Delta\sigma_e$ and the stress range due to Q_e. This ratio is shown in FIGURE 10 [22] in terms of span length and for three different countries.

It is very suprising to note the small difference in this correction factor even between countries having a different traffic pattern. The fact that the "average" truck might be heavier in one country than in another is reflected by the numerical value of the equivalent weight. Based on these observations, it now appears possible to define a harmonized traffic model for fatigue design using the correction factor applied to the equivalent weight.

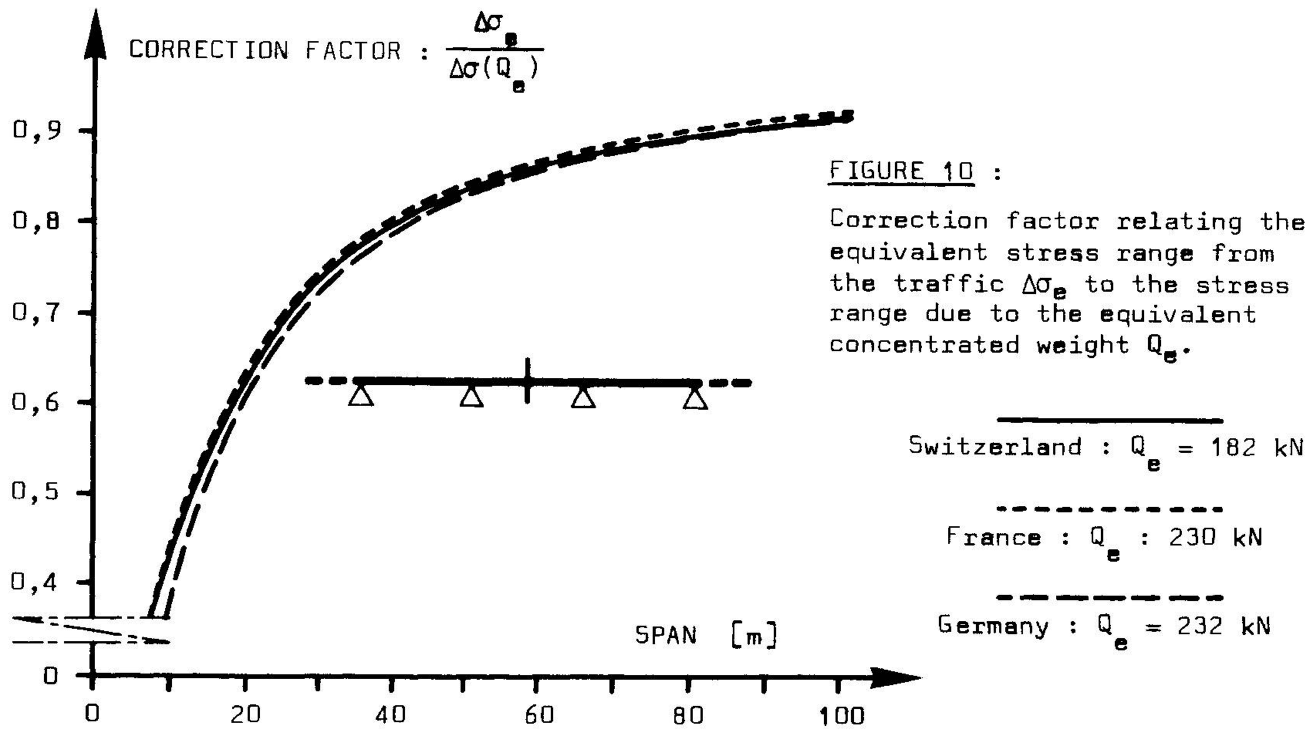


FIGURE 10 :

Correction factor relating the equivalent stress range from the traffic $\Delta\sigma_e$ to the stress range due to the equivalent concentrated weight Q_e.

4. METHODS OF EVALUATION

In addition to the basic parameters discussed in the previous section, safety considerations or margins of safety must be introduced for the evaluation of the remaining fatigue life. Some basic concepts may be distinguished [23] [24] [25].

Level 1 : Deterministic approach.

In this approach fixed values are assigned to all parameters, for example mean or fractiles. When the fatigue strength curves are extended below the fatigue limit an analytical solution using the equivalent stress range concept is possible. In any case one should always proceed in steps in order to identify the effect of the individual assumptions on the resulting fatigue life estimation.
A safety factor can be introduced on this life estimation depending on the degree of precision of the individual parameters introduced.

Level 2 : Pseudo-probabilistic approach.

This has recently been introduced by the Swiss steel specification and the UIC recommendation where the scatter of the fatigue strength is represented by a log-normal probability density function. Also, the equivalent stress range has a log-normal distribution assigned to it. The effects of load and strength are thus "separated" by means of counting method and cumulative damage rule.

Usual safety considerations, as developed for ultimate strength design, can be used by introducing a safety index β. Incidentally, this is only possible if the log-normal distribution of the fatigue strength, which was originally obtained on the horizontal (number of cycles) axis, is transformed into the vertical (stress range) axis. However, the result may still be expressed in terms of life. The major problem remains the calibration with experience in order to define the numerical value of β.

Level 3 : Probabilistic approach.

All parameters must be introduced with their statistical distribution. A major problem in the analytical methods resides in the fact that strength is not independant of stress spectra. Cumulative damage rules have to be expressed in a different way and the result will be in terms of probability of survival.

Research to establish these analytical methods is under way [26] and numerical procedures using for example Monte Carlo simulations also seem possible. In order to reduce the number of parameters that have to be introduced in such computations, the results shown in paragraph 3.8 might be of interest.

5. CONCLUSIONS

This paper summarized the reasons and circumstances which might lead to an evaluation of the remaining fatigue life. The basic parameters needed for such an evaluation have been enumerated and discussed. The main purpose of the evaluation procedure is the rating of the structure. Unfortunately, no clear or agreed upon procedures exist and more work is urgently needed to establish such methods. Nevertheless, a certain number of ideas might be retained.

1.- When a crack is found in a structure, this generally indicates that many more cracks are present. Repair and retrofit procedures must be established using for example fracture mechanics analysis. It has to be stressed that the remaining fatigue life is generally very short once the cracks are easily visible, and thus found.

Repairs are often very costly, hence, small span structures might most economically be replaced by adequately designed structures. Long span structures very rarely suffer fatigue damage in the principal structural elements, unless cracks in the secondary elements have grown into them. Whenever retrofit of a superstructure is needed one should also try to reduce impact factors by, for example, changing the load path of directly introduced loads.

2.- The rating of a structure obviously needs a clear evaluation procedure and the necessary information on the basic parameters. If the calculated remaining life is negative, then two possibilities exist : one, the assumptions are too conservative (impact factor, stresses in a highly redundant structure, loads) or two, the problem is real, in which case fatigue damage is very probable.

If the calculated remaining life is positive, an appropriate safety factor is needed (level 1 or 2) on the remaining life (not stress) of each element whilst taking into account its importance for the entire structure. In other words, the redundancy of the structure becomes a significant factor to judge the importance of possible cracking ; for example, a multibeam bridge will be not as critical as a two girder bridge. Based on this, inspection procedures and intervals have to be defined.

3.- The rating of a given set of structures places less importance on the choice of the safety margin since the primary goal of the evaluation is to establish priorities for inspection of replacement.

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