# FATIGUE ASPECTS IN THE DESIGN DETAILS OF STEEL BRIDGES

**Dr. P. Suryanarayana**
**Asst. Prof., Civil Engineering,**
**M.A.C.T. Bhopal, India.**

## ABSTRACT

Steel bridges which are designed adequately by the strength and deflection criteria are to be further checked for the length of the service life of various components (also known as details). These components such as cleats, gussets, bolts, rivets, welds and stiffeners are to be individually assessed for their service life under fluctuating stresses caused by moving vehicles with varying axle weights. The details are classified in accordance with their importance. This paper describes the identification of the details to be assessed (hot spots) in steel bridges so that these can be strengthened to achieve full service life.

## 1. INTRODUCTION

The structural design of steel bridges involve proportioning of the cross section so that the stresses developed under self weight plus traffic or dynamic loads are below permissible limits. Additionally it is necessary to check for serviceability of the bridge which means that the total deflection under the worst combination of loads as well as the differential deflection due to live load is not excessive. Finally it is necessary to check the individual components of the bridge so that it performs satisfactorily throughout its designated life. Generally bridges are designed for a service life of 120 years (Mallet, 1991). No components of the bridge should fail before the designated service life. If any detail of the bridge shows premature signs of distress it is necessary to assess the remaining service life and strengthen the detail to achieve the designated service life.

## 2. BRIDGE DETAILS

Codes of practice (such as BS 153, BS 5400) identify and classify the bridge components whose service life is to be assessed in order to ensure that the bridge lasts for the 120 years envisaged in design. These components are designated as A,b,........,F1,F2,G etc. in the descending order of strength. However the code listings are not exhaustive and the designer has to rely on intuition and judgement (Ogle, 1981). These details such as cleats, gussets, bolts, rivets, welds and stiffeners are vulnerable to failure caused by fluctuating stresses mainly due to moving traffic. The failure of a structural detail under alternating stress at a level lower than static failure stress is said to be caused by fatigue.

## 3. FATIGUE FAILURE OF BRIDGE DETAILS

If a detail is subjected to thousands of cycles of alternating stresses such that the stress range (algebraic difference between the maximum and minimum stress) is $f_r$, the failure condition is given by

$$N\,(f_r)^m = C \qquad (1)$$

where N is the number of cycles at which failure takes place, m is an exponent and C is a constant. Generally there is a sharp increase in the value of m when the value of N changes from `low' (N = 10000 to half million) to `normal' (N = half million to ten million) to `high' (above ten million) (Mallet, 1991). The fatigue life is a function of the stress range alone, no dead load stresses contributing to fatigue risk.

The fatigue equation 1 predicts the service life for a given stress range. In order to evaluate the service life under different values of $f_r$ caused by different vehicles Palmgren-Miner summation is used. The bridge is traversed by `k' types of vehicles. If $n_i$ is the number of passes made by vehicle of type i and $N_i$ is the number of passes causing failure if this vehicle alone traverses the bridge, failure occurs when

$$(n_1 / N_1) + (n_2 / N_2) + \dots\dots\dots + (n_i / N_i) + \dots\dots\dots + (n_k / N_k) = 1 \qquad (2)$$

In order to apply Palmgren-Miner summation it is necessary to obtain $n_1, n_2 \dots\dots n_k$ by a traffic census. If exhaustive traffic data is available more sophisticated methods such as `rainflow' method and `reservoir' method can be adopted (Mallet, 1991).

## 4. FATIGUE RISK OF BRIDGE DETAILS

The fluctuating stresses due to traffic load leads to fatigue risk causing different types of failure in different details. Friction grip bolts are subjected to loss of gripping friction and are also subjected to prying forces. Ordinary bolts are subjected to fretting forces. Studs in composite beams (with concrete deck slabs) offer reduced bearing and uplift resistance. Ward (1981) suggests estimates for loss of strength in bolted connections and provides design formulae. Johnson (1981) makes similar recommendations for studs. For welded joints the following points are to be noted: (i) Tack welding should be avoided as this type of welding alters the chemical structure of steel and also causes brittleness in the surrounding steel (Mallet, 1991), (ii) Plug welds may be permitted provided relevant standards are followed (Ward, 1981), (iii) Full penetration butt welds are most preferable, (iv) Fillet welds made by submerged arc process show twenty percent increase in throat thickness as compared with other methods, (v) If hybrid connections are used, welding should be completed well before the tightening of bolts. Design curves and formulae are listed by Ward (1981).

Cullimore (1981) describes fatigue failure of threaded bolts due to fretting (continual, to and fro motion in an annulus at the outer edge of pressure area). In this form of failure cracks are initiated in the annulus of the pressure area and spread across the gross section of the plate. Cullimore (1981) proposes expressions for plate thickness and width as functions of bolt diamter to avoid fatigue failure. He has also shown that fatigue strength cannot be increased by merely increasing the number of bolts. Ogle (1981) discusses fatigue failures of web stiffeners and compression flange plates. The estimation of stresses in these members under different types of axle loads is a time consuming process. Traffic studies in U.K. have shown that stresses due to different axle loads can be represented as fractions of stresses caused by a standard fatigue vehicle. Vehicle loads in other countries are not amenable to such a simplification.

From the foregoing description it can be seen that fatigue risk analysis is a lengthy process because of the following reasons : (i) Palmgren-Miner summation, rainflow method and reservoir method can yield satisfactory results only if extensive traffic census is available. (ii) The stress range $f_r$ is to be calculated for each of traversing vehicle. In Europe there are 27 types of vehicles with axle loads large enough to cause significant value of stress range. (iii) The stress range is to be calculated for several different components of the bridge. Though it is a daunting task to perform these calculations, fatigue risk analysis cannot be avoided. Halse (1981) points out that a 500 mm deep steel section which can sustain a load of 15t without permanent deformation and 25.8t. before fracture may fail at less than 7.3 t load applied five million times. If this section has a welded attachment the failure load can be as low as 2.25t.

However fatigue analysis of bridge components is less complex for welded joints as compared to other types. Since welding is rapidly replacing other types of connections mentioned earlier, it is possible to develop an analysis for details of welded bridges (Gurney, 1981). The analysis is based on limit state approach.

## 5. BRIDGES WITH WELDED JOINTS

Extensive fatigue data (S-N curves representing eq.1, effect of type of welding, effect of residual stresses) is available for welded details and is tabulated by Gurney (1981). Welded joints are classified from A to G in the order of descending fatigue strength. (At 2 million cycles class A joint has strength of 240 MPa while class G has 52 MPa). The joints are analysed by limit state approach incorporating suitable partial safety factors. Since the fatigue strength depends only on stress range and not on static strength of parent material or the value of maximum and minimum stress, the rules can be applied for steels with yield strength ranging between 232 to 850 MPa. Values of exponent m for different classes of details are given. These values take into account the effect of residual stresses, stress cycles with peak and trough both compressive, cycles with compressive-tensile reversal and the effect of material thickness. In BS 5400 welded joints are classified into six major groups each listed with design parameters applicable to that particular group. The initiation and propagation of fatigue cracks in these six details is described to aid the designer.

## 6. DESIGN PROCEDURE FOR WELDED BRIDGE

The foregoing description makes it possible to list the steps in complete design (which includes assessment of fatigue life) for welded bridges. (i) The structural design is done and the section is proportioned. All details are designed using welded connections throughout. (ii) Serviceability (deflection) check is made. (iii) The details along the span are listed and classified A to G depending on type of joint (fillet, butt, plug), method of welding (simple arc welding, submerged arc), relative orientation of welded plates and discontinuities in welding. (iv) The stress range caused by a typical vehicle and the corresponding failure cycle number $N_i$ is obtained. The exercise is repeated to all types of vehicles that traverse the bridge. (v) Apply Palmgren-Miner summation and assess fatigue risk. If the detail fails, the weld is to be strengthened by increasing weld thickness, adopting closer weld spacing or replacing the joint by adding additional members.

## 7. CONCLUSIONS

The fatigue risk in bridge components can be assessed by available methods (i) to classify the details and provide design information for each detail, (ii) to assess the fatigue life of each detail under a given traffic pattern by calculation of stress ranges caused by different types of vehicle and apply a summation formula and finally (iii) to modify the bridge detail to increase the fatigue life to the service life envisaged in design.

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