FATIGUE AND REMAINING LIFE ASSESSMENT OF STEEL BRIDGES MORE THAN 50 YEARS OLD

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Abstract
The present bridge network in Quebec includes approximately 9000 bridges of different types and represents a 9 billion dollars asset. Many bridges show signs of deterioration well before the end of the design life, supposedly between 50 to 100 years. Truck loading has increased by more than 40% in the past 40 years, and bridges have not been reinforced in order to comply with the new codes. The accumulated fatigue and remaining life of highway bridges will be reviewed in this paper. Railroad bridges are excluded because their loading analysis uses a different approach.

The condition of steel bridges in full service for more than 50 years, which is the average age of most of bridges in operation in Canada, must be evaluated in a realistic way. Fatigue, severe climatic conditions such as temperature difference between hot and cold seasons, de-icing salt and corrosion are deterioration factors. At low temperatures, steel loses its ductility, thus increasing the risk of brittle fracture. Steel used 50 years ago was poorly deoxidized during pouring, and consequently fatigue cracks can propagate more easily through the impurities.

1. Introduction
The level of deterioration of the road network, specially bridges, in Quebec requires urgent repairs. About 4700 bridges are managed by the Ministère des Transports du Québec. Other bridges are controlled by the Ministère des Forêts du Québec, Hydro Québec, Transports Canada, la Voie Maritime du Saint-Laurent and railroad companies.

Live loads have increased 40% during the last 40 years. Heavier trucks and traffic produce stresses in steel structures that are far superior to those expected in the original design. Fatigue can cause brittle fracture of steel members. Most structures have not been reinforced to comply with the new codes. Fatigue occurs at stress values below the static
strength of steel, and unfortunately brittle fracture collapses due to fatigue often produce a loss of human lives.

In Quebec, the average value of the structural index is 68%. Bridges having a structural index between 40% and 75% need reinforcement. Bridges with a structural index below 40% are considered in poor condition and may require replacement [1]. Fatigue failure increases with values of the structural index larger than 50%. The situation is similar in most industrialized countries. In the United States, 40% of the 650,000 bridges are considered to be structurally deficient and require complete replacement or major rehabilitation [2].

Northern countries face severe climatic conditions that produce additional deterioration factors for steel bridges, including temperature effects and corrosion accelerated by de-icing salts. Presently, Quebec does not have a standard procedure for fatigue and remaining life assessment of steel bridges older than 50 years. A better understanding of the real behavior of steel bridges will help establishing an assessment procedure and guiding engineers when deciding between reinforcement and replacement.

2. Fatigue of steel bridges

Most fatigue damages in a bridge are caused by the passages of single trucks. These can exceed 100 million in a 50 to 100 year life, but are often much less. The effective stress range rarely exceeds 35 MPa and usually ranges between 5 and 20 MPa. However, this is enough to cause fatigue fracture over the years as proven by recent collapses.

Evaluation procedures are intended to reflect the actual fatigue conditions that occur in a structure. They generally involve three steps:

1. Calculate the variable-amplitude stress spectrum caused by the actual loading.
2. Relate this variable-amplitude stress spectrum to an equivalent constant-amplitude stress by some cumulative approach.
3. Compare the resulting applied stress parameter with a fatigue strength curve in order to get the fatigue life or show that the applied stress is below an allowable stress value.

Fatigue behavior and remaining life assessment is uncertain still today. It is difficult to evaluate the exact number of load cycles and the effects of truck volume and weight increases. Truck traffic history is often unknown, original drawings and specifications are not available, and maintenance history, truck superposition, lateral distribution and future traffic are parameters difficult to establish accurately. Assessment is even harder when the structure has been repaired or reinforced.

Due to the increased risk of fatigue failure, many bridges require reinforcement, rehabilitation or replacement. It is not easy to decide what to do when the real condition
of the bridge is unknown. Weight restrictions are imposed to bridges under higher risks, especially bridges located on secondary roads, often the oldest and most affected by the passage of heavier trucks. There are more than 300 000 bridges older than 50 years in the United States and approximately 20 000 in Canada.

Chapter 14 on Bridge Evaluation of the Canadian Standard CAN/CSA S6-00 [3], states that fatigue has to be evaluated using proper methods. However, no method is recommended. The procedure proposed by Moses [4] and latter published by the American Association of State Highway and Transportation Officials (AASHTO) in 1990 is a main reference in this matter. It is based on an extensive series of tests carried out by the National Cooperative Highway Research Program (NCHRP). These tests were done at normal temperature, ignoring corrosion and the effects of reinforcement for older structures. Recent bridge collapses were due to corrosion of steel members and connections.

According to AASHTO, the remaining safe life is given by:

\[
Y_f = \frac{f \times 10^6}{T_d C (R_s S_r)^3} - a
\]

where

- \( f \) Remaining life factor.
- \( K \) Constant for detail category as per AASHTO.
- \( T_d \) Daily truck traffic in shoulder lane.
- \( C \) Cycles per truck passage.
- \( S_r \) Nominal stress range produced by truck load in ksi.
- \( R_s \) Reliability factor.
- \( a \) Age of bridge in years.

Note that Equation 1 uses imperial units.

### 3. Fatigue strength of steel members

All temporary or permanent structures must comply with two basic requirements. They must have enough strength so they do not collapse (ultimate limit states) and they must behave satisfactorily under service loads (serviceability limit states). Ultimate limit states are always verified using factored loads, except fatigue which must be verified under service loads.

Fracture of a member under cyclic loading is produced by stresses due to the passage of trucks. Microscopic cracks initiate at some discontinuity in the crystal structure of steel.
or at friction points between members. The fatigue process has three phases: 1) crack initiation, 2) crack growth and 3) failure.

Several factors might accelerate crack initiation: corrosion, residual stresses, geometric discontinuities, stress concentrations, eccentricities in connections, loading speed, and relative displacements between members.

Fatigue strength is generally higher for bolted connections than for welded connections [5]. Lack of redundancy is the main reason for this difference. In bolted connections, stresses are flowing more easily from one member to another. Plate girders often use stiffeners in order to prevent web buckling, and welding the lower end of the stiffener to the tension flange of the girder is not recommended because of fatigue risks.

The fatigue strength of a member is defined as the number of load cycles, $N$, applied before the fracture for a given stress range $\sigma_a$. The stress range is the algebraic difference between maximal and minimal stresses at a given point of the member where tension stresses are considered positive and compression negative:

$$\sigma_a = f_w = \sigma_{\text{max}} - \sigma_{\text{min}}$$

The fatigue life curves represent the relationship between stress range, type of detail and number of cycles (Figure 1). Constant amplitude thresholds under which a crack will not grow are shown by dash lines.

![Figure 1: Fatigue constants for various detail categories [6]](image-url)
Miner’s Law [7], originally proposed by Palmgren [8], is used in most procedures for relating variable amplitude fatigue to constant amplitude fatigue. Extensive studies done by NCHRP of simulated bridge members showed that the scatter in predicting fatigue life is not large using this method. The damage fraction that results from any particular stress range level is assumed to be a linear function of the number of cycles that takes place at the stress range. The total damage from all stresses that are applied to the detail is equal to the sum of all damage fractions. Its value has to be less than one in order to be satisfactory:

\[ \sum \frac{n_i}{N_i} \leq 1.0 \]  

(3)

4. Research on steel bridge fatigue

Between 1962 and 1995, extensive studies done by NCHRP have produced the knowledge and the data base used today in many codes to evaluate fatigue damage in steel bridges. NCHRP is administrated by the Transportation Research Board (TRB) and is sponsored by AASHTO.

NCHRP report No. 229 [4] presented an assessment method for steel bridge fatigue damage and remaining life. Before this publication, procedures did not reflect real fatigue conditions. Remaining life assessment was not covered properly and methods were not suitable for bridges already in service. Results of the study were incorporated in the Bridge Evaluation Manual published by AASHTO in 1991. CAN/CSA S6-00 also uses results from the NCHRP research, although climatic winter conditions in Canada are quiet different from those used during the laboratory tests.

Secondary members such as diaphragms and bracing elements are susceptible to cause secondary bending caused by out of plane effects, namely web distortions which can accelerate fatigue cracks. These cracks are located typically where a floor beam is attached to a connection plate welded to the web of a girder. Under the passage of traffic, the floor beam rotates and, as rotation occurs, the bottom flange tends to lengthen and the top flange tends to shorten. Lengthening of bottom flange is not restrained because it is pushing into the web of the girder, which is flexible in this out of plane direction. However, because the top flange of the girder is restrained by the deck slab, shortening of the top flange can only be accommodated by deformation within the gap at the top of the connection plate. This type of deformation is shown in Figure 2.
5. Canadian Standard CAN/CSA S6-00

Standard CAN/CSA S6-00 states that where there are fatigue-prone details or physical evidence of fatigue-related defects, the bridge and affected members shall be assessed for fatigue and remaining life using appropriate methods. However, no method is recommended.

With fatigue, the most important stress parameter is the magnitude of the cyclic variation. Fatigue failure does not occur unless part of the stress cycle involves tension. However, caution is necessary before dismissing the possibility of fatigue failure in regions of nominal compressive stresses, because unexpected tensile stresses may occur due to residual stresses or local tri-axial stress conditions.

5.1 Fatigue due to live loads

CAN/CSA S6-00 states that for load-induced fatigue, each detail of a bridge, except in decks, shall satisfy the following requirement:

\[ 0.52 f_{sr} \leq F_{sr} \quad (4) \]

where \( f_{sr} \) is the calculated stress range at the detail due to passage of a CL-625 truck with five axles and a weight of 625 kN, and \( F_{sr} \) is the threshold stress range. The fatigue stress range resistance of a member or a detail shall be calculated from the following equation:

\[ F_{sr} = \left( \frac{\gamma}{N_e} \right)^{\frac{1}{2}} \geq 0.5 F_s \quad (5) \]
where $\gamma$ is the fatigue life constant pertaining to the detail category, and $N_c$ is the number of cycles determined as follows:

$$N_c = 365yN_d(DJMC_f)$$

(6)

where $y$ is the design life equal to 75 years unless otherwise specified and $DJMC_f$ is the single lane average daily truck traffic:

$$DJMC_f = p(DJMC)$$

(7)

Longitudinal bending moments associated with the fatigue limit state are calculated as follows:

$$M_g = F_m \times M_{avg}$$

(8)

$$M_{avg} = \frac{M_t}{N}$$

(9)

where

- $M_g$: Longitudinal bending moment in the beam.
- $M_{avg}$: Average bending moment calculated by distributing uniformly the total moment in the section of the bridge, divided by the number of beams in the section.
- $F_m$: Amplification factor considering the cross section variation and maximal intensity of longitudinal moment related to the average intensity of the longitudinal moment.
- $M_t$: Maximum moment for a single truck loading at the section evaluated.
- $N$: Number of beams along the width of the deck.

6. Analysis of bridge fatigue in other countries

The British Standard for bridge fatigue damage assessment treats compression loading cycles in the same way as tension cycles. The European Convention for Construction Steelworks (ECCS) treats compression and tension stresses equally in welded members. In America, only tension members are assumed to develop fatigue cracks, hence only tension cycles are verified.

In Japan, most steel bridges were built after 1960 [10]. Fatigue assessment was compulsory only for railroad bridges until 1989 when many fatigue cracks were observed on steel road bridges, thus forcing the government to revise the standards. Today, some bridges in urban areas of Japan carry up to 40 000 trucks daily on two lanes. Research
work has developed rapidly in order to build a reliable database. Standards were updated to match those existing in United States and Europe.

In United States, a study of the Arkansas River Bridge [11] revealed 109 fatigue cracks. It is estimated that 80% of all steel bridges in this State have developed fatigue cracks, many of which caused by out-of-plane effects in the gap above web stiffeners connected to a diaphragm or a brace.

The State of Arkansas tried two methods for repairing damages on stiffeners. The first method consisted in cutting the stiffener in order to provide more flexibility to the connection, the second one in adding another stiffener behind the existing one in order to reinforce and give more rigidity to the connection. The latter method performed better.

According to Tsiatas [12], the AASHTO fatigue assessment procedure produces non-realistic life remaining results. Tsiatas evaluated four steel bridges with a redundant structure using principles of linear elastic mechanics and found remaining life longer than that obtained by the AASHTO procedure, 6 to 10 times longer for some bridges.

In December 2000, a section of the roadway in the Hoan Bridge on Highway I-794 began to sag visibly. Cracks were detected in the three girders supporting the spans. Brittle fracture of the steel occurred and the most critically damaged section of the roadway was demolished using explosives. It was concluded that the fracture originated in the lower shelf-plate joint assembly and traveled the height of a 10-foot steel “I” girder in a single cycle. The Hoan Bridge was carrying 36,000 vehicles daily and raised the need for extended inspection on bridges with similar details. Evaluation of these bridges is difficult due to the uncertain values of many factors involved in crack development.

A study carried out in 1998 at the École de technologie supérieure revealed that steels used over 30 years ago were poorly deoxidized at the time of pouring. As a result, there were large quantities of impurities increasing the susceptibility for a brittle fracture, given that fatigue cracks can propagate more easily through the impurities. This study was conducted following the collapse of the main girders in the Bécancour Bridge.

7. Conclusion

Current procedures do not take in consideration some important factors involved in recent bridge failures. The authors propose to evaluate more realistically the fatigue damage and remaining life of bridges over 50 years old. An improved procedure will be set as a final stage of their proposed work.
8. References